AD-A142 621 NATIONAL DAM INSPECTION PROGRAM UNION POND DAM (CT 1/2
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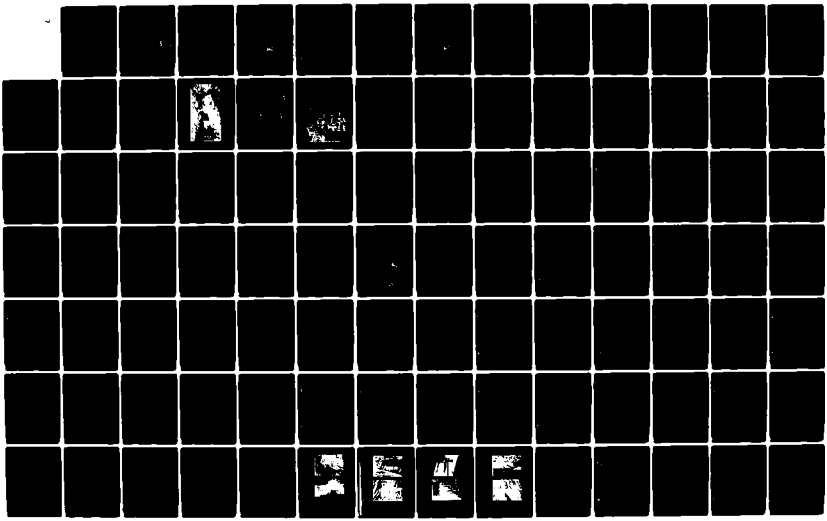

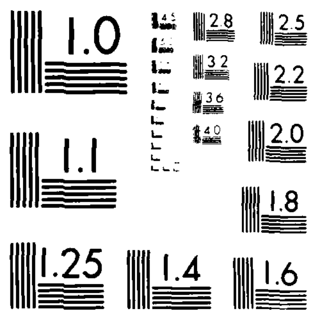

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REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER CT 00013	2. GOVT ACCESSION NO. AD-A142 621	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Union Pond Dam, Upper Connecticut River Basin ~~Phase I Inspection Report~~ NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE Feb. 1979
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14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED
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16. DISTRIBUTION STATEMENT (of this Report)

APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

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17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY, Pond, Union

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is a concrete gravity structure with the spillway constructed in an "L" shape. The total length of the dam is approximately 590 ft. including the earth dike. The top of the dam is approximately 33 ft. above the bed of the Hockanum River. The spillway is a broad crested compound weir of trapesoidal cross-section consisting of an outer concrete shell over an inner earth and rubble core. In 1972 No. 8 reinforcing bars grouted into 2 inch diameter holes 20 ft. long and spaced at 10 ft. intervals were installed through the top of the old dam, probably in an attempt to stabilize the upper portion of the present dam.

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UPPER CONNECTICUT RIVER BASIN

MANCHESTER , CONNECTICUT

UNION POND DAM
CT 00013

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

FEBRUARY 1979

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED-E

SEP 10 1979

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

Inclosed is a copy of the Union Pond Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. The report is based upon a visual inspection, a review of past performance, and a preliminary hydrological analysis. A brief assessment is included at the beginning of the report.

The visual inspection conducted at the site has revealed that the concrete in the downstream face of the spillway has suffered serious deterioration. Due to this, the stability of the structure appears to be marginal based upon existing data. In addition, the preliminary hydrologic analysis has indicated that the spillway capacity for the Union Pond Dam would likely be exceeded by floods greater than twenty-eight percent of one-half the Probable Maximum Flood (1/2 PMF), the test flood for spillway adequacy. Our screening criteria specifies that a dam of this class which does not have sufficient spillway capacity to discharge fifty percent of the PMF, should be adjudged as having a seriously inadequate spillway. As a result of the concerns of the stability of the dam in conjunction with the serious inadequacy of the spillway, the dam has been assessed as unsafe until corrective measures are completed.

It is recognized that the owner has engaged the services of a professional consulting engineer to investigate the deficiencies of the dam, including those previously mentioned, as recommended in the draft report previously forwarded to Commissioner Pac's office. It is recommended that based upon this investigation appropriate remedial mitigating measures should be designed and completed within 12 months of this date of notification. In the interim a detailed emergency operation plan and warning system should be promptly developed. During periods of unusually heavy precipitation, round-the-clock surveillance should be provided.

NEDED-E
Honorable Ella T. Grasso

I have approved the report and support the findings and recommendations described in Section 7, with qualifications as noted above. I request that you keep me informed of the actions taken to implement these recommendations since this follow-up is an important part of the non-Federal Dam Inspection Program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. This report has also been furnished to the owner of the project, the Town of Manchester, 41 Center Stret, Manchester, Connecticut 06040, ATTN: Mr. Jay Giles, Public Works Director.

Copies of this report will be made available to the public, upon request to this office, under the Freedom of Information Act, thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for the cooperation extended in carrying out this program.

Sincerely,

M. B. Scheider

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

UPPER CONNECTICUT RIVER BASIN
MANCHESTER , CONNECTICUT

UNION POND DAM
CT 00013

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

FEBRUARY 1979

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	UNION POND DAM
Inventory Number:	CT 00013
State Located:	CONNECTICUT
County Located:	HARTFORD
Town Located:	MANCHESTER
Stream:	HOCKANUM
Owner:	TOWN OF MANCHESTER
Date of Inspection:	NOVEMBER 27, 1978
Inspection Team:	PETER HEYNEN
	CALVIN GOLDSMITH
	GONZALO CASTRO

The dam is a concrete gravity structure with the spillway constructed in an "L" shape. The total length of the dam is approximately 590 feet including the earth dike. The top of the dam is approximately 33 feet above the bed of the Hockanum River. The spillway is a broad crested compound weir of trapezoidal cross-section consisting of an outer concrete shell over an inner earth and rubble core. In 1972 No. 8 reinforcing bars grouted into 2 inch diameter holes 20 feet long and spaced at 10 foot intervals were installed through the top of the old dam, probably in an attempt to stabilize the upper portion of the present dam. The spillway crest is four feet below the top of the dam abutments. There are four outlets from the dam. A 42 inch low level outlet is at the right end of the spillway which is referred to on the existing 1901 plan as the "old waste gate". At the extreme left end of the spillway, there are two 2'x3' intermediate level sluice gates through the dam. The left gate is operational while the right floor stand is disconnected from the gate and hence will not function.

The fourth outlet is in the gatehouse at the extreme left end of the dam between the left dam abutment and the earth dike. The outlet feeds a cast iron conduit nine feet in diameter. The conduit runs under the road and flows back into the river further downstream. The gate to the conduit is presently inoperable. To the left of the gatehouse is the earth dike, which is approximately 175 feet long and has an average crest elevation of 146.7.

Based upon the visual inspection and its past performance, the dam appears to be in poor condition. The stability of the structure appears to be marginal based on existing data, and the downstream concrete facing of the

spillway is heavily deteriorated. The condition of the dike appears good, however the gatehouse adjacent to the dike and dam, is partially demolished. The condition of the 9 foot conduit and the gate controlling it are questionable and warrant attention. There are other minor areas requiring attention as well.

Based upon the size (Small) and hazard classification (High) of the dam in accordance with Corps of Engineers Guidelines, the Test Flood will be equivalent to one-half the Probable Maximum Flood (PMF). Peak inflow to the pond is 31,000 cfs; peak outflow (Test Flood) is 30,500 cfs with the dam overtopped 3.9 feet. Based upon our hydraulics computations, the spillway capacity is 8400 cubic feet per second (cfs), which is equivalent to 28% of the Test Flood.

It is recommended that further studies be undertaken to perform a more refined hydraulic/hydrologic study to determine the best way to increase the ability of the spillway to pass a greater percentage of the Test Flood, and to increase the overall discharge capacity of the facility, including the gates.

A registered professional engineer qualified in dam engineering should immediately investigate the stability of the dam, and develop recommendations to adequately increase the dam stability and eliminate seepage through the dam.

The condition of the 9 foot diameter conduit should be investigated and consideration given to renovating and maintaining it as another low level outlet to be used in times of high water. Should the owner decide to seal off the conduit, it should be done permanently, and as close to the gate as possible.

A repair scheme to renovate the downstream concrete surfacing should be included in the recommendations. Other areas requiring attention include the damaged gatehouse, the inoperable right sluice gate, trees growing on the earth dike, and the contact seeps at the right abutment. An operations and maintenance plan should be instituted as well.

The recommendations discussed above and in Section 7, should be instituted immediately upon the owner's receipt of this report, while the remedial measures, also in Section 7, should be instituted within one year of the owner's receipt of this report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

Edgar B. Vinal, Jr., P.E.
Senior Vice President
Cahn Engineers, Inc.

This Phase I Inspection Report on Union Pond Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Joseph A. McElroy

JOSEPH A. MCELROY, MEMBER
Foundation & Materials Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Joseph W. Finegan, Jr.

JOSEPH W. FINEGAN, JR., CHAIRMAN
Chief, Reservoir Control Center
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized t[illegible] the reported condition of the dam is based on observat[illegible]ns of field conditions at the time of inspection along wi[illegible] data available to the inspection team. In cases wh[illegible] the reservoir was lowered or drained prior to inspecti[illegible] such action, while improving the stability and safety of [illegible]e [illegible]am, removes the normal load on the structure and may [illegible]scure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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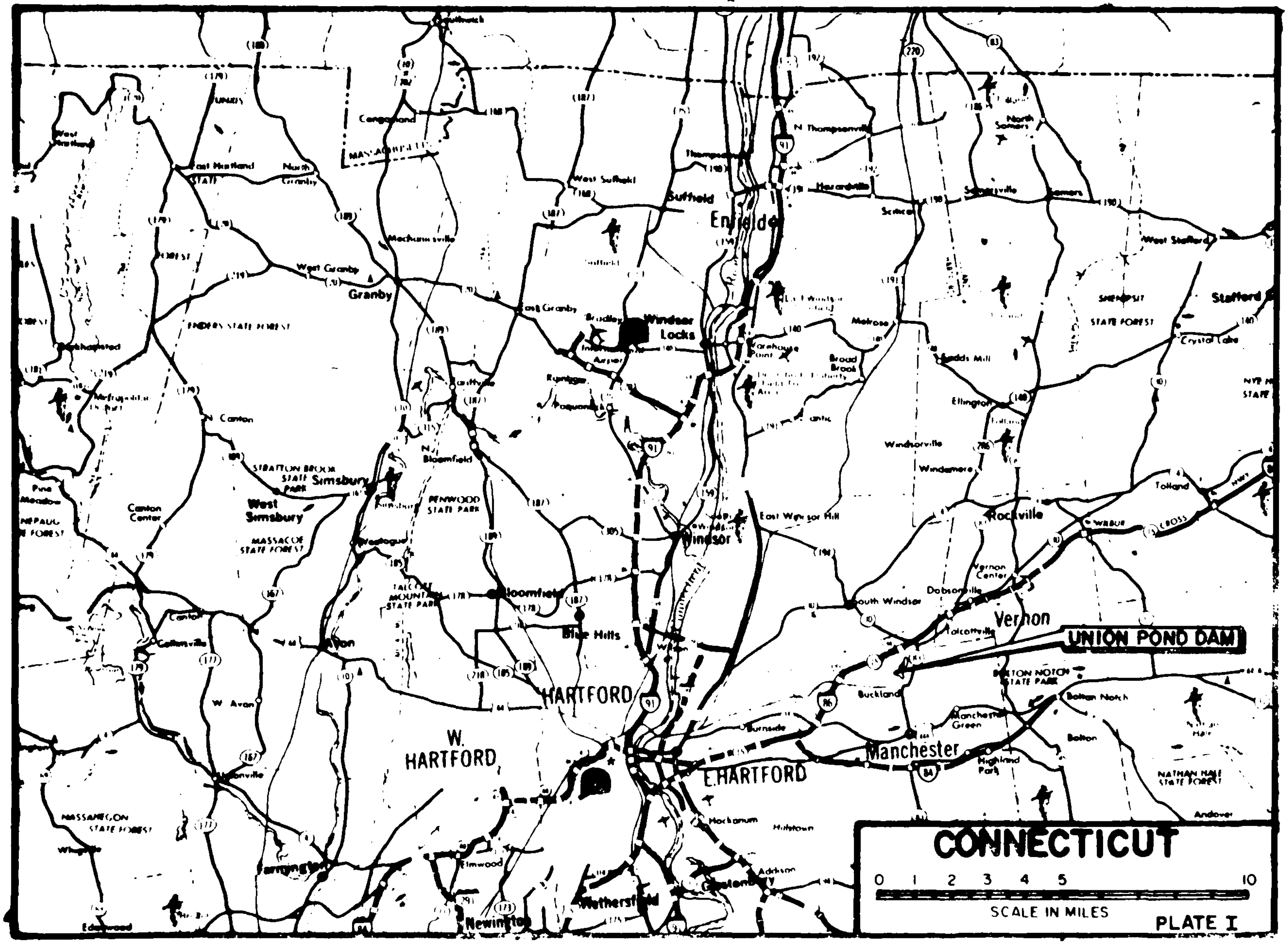
SECTION 7: ASSESSMENT, RECOMMENDATIONS & REMEDIAL MEASURES

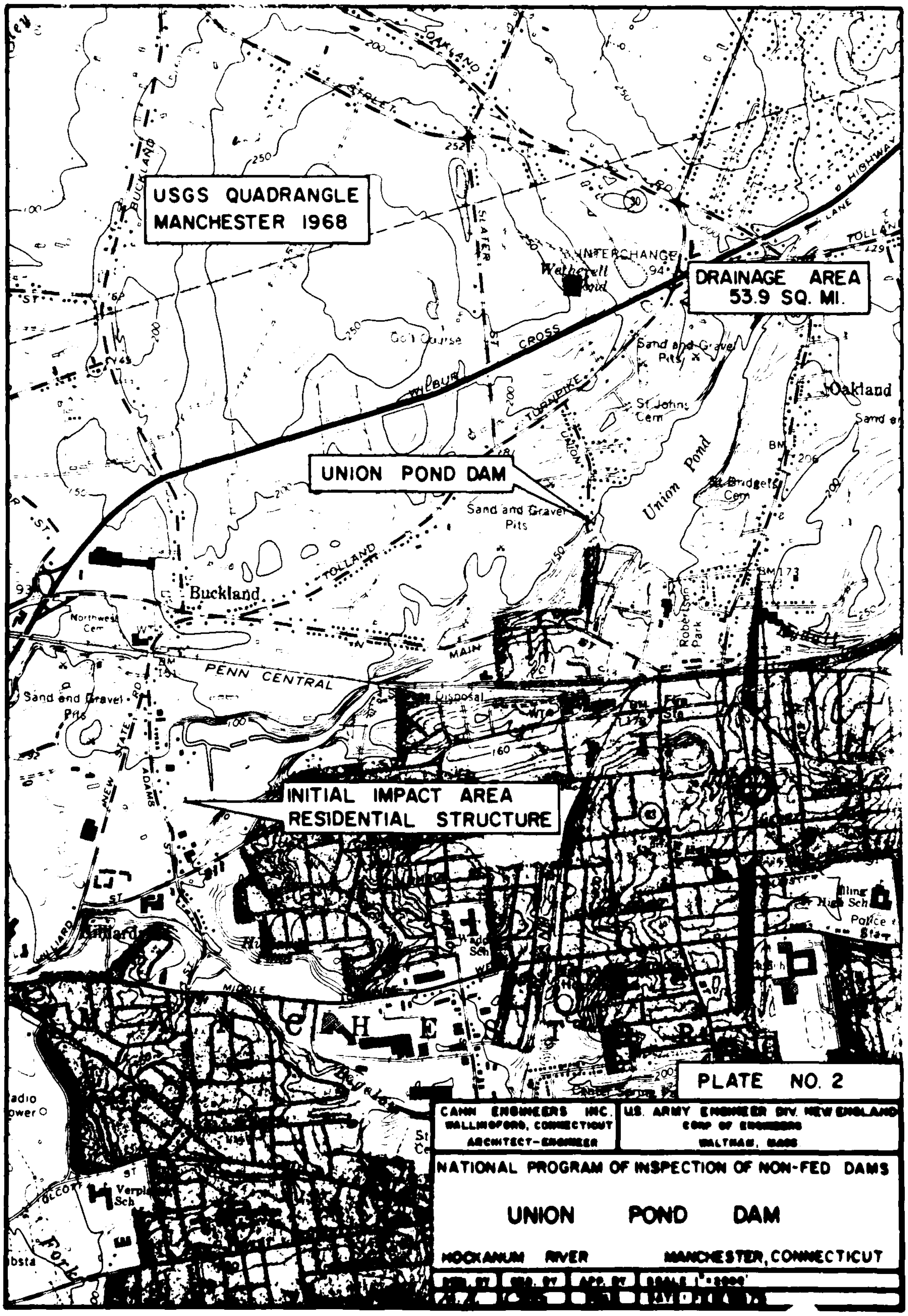
APPENDIX

OVERVIEW PHOTO

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS. CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT - ENGINEER	NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS	[illegible]	[illegible]	DATE [illegible] CE # PA[illegible]



UNION POND DAM
CONNECTICUT
0 1 2 3 4 5 10
SCALE IN MILES
PLATE I
Enfield
Windsor Locks
Suffield
West Suffield
Granby
West Granby
East Granby
Simsbury
West Simsbury
Canton Center
N. Canton
Bloomfield
Blue Hills
HARTFORD
W. HARTFORD
E. HARTFORD
Manchester
Vernon
Rockville
South Windsor
Windsor
East Windsor Hill
Broad Brook
Ellington
Tolland
Stafford
Crystal Lake
West Stafford
Somersville
Hazardville
N Thompsonville
North Somers
Bolton Notch
Bolton
Buckland
Burnside
Manchester Green
Highland Park
Hockanum
Andover
Avon
W. Avon
Collinsville
Edgewood
Elmwood
Newington
Wethersfield
Glastonbury
Windsorville
Windermere
Talcottville
Dobsonville
Vernon Center
Melrose
Warehouse Point
Congamond
Mechanicsville
Poquonock
Rainbow
West Hartland
East Hartland
North Granby
STRATTON BROOK STATE PARK
PENWOOD STATE PARK
TALCOTT MOUNTAIN STATE PARK
MASSACOE STATE FOREST
NASSAHEGON STATE FOREST
SHENIPSIT STATE FOREST
NATHAN HALE STATE FOREST
BOLTON NOTCH STATE PARK


USGS QUADRANGLE
MANCHESTER 1968
DRAINAGE AREA
53.9 SQ. MI.
UNION POND DAM
INITIAL IMPACT AREA
RESIDENTIAL STRUCTURE
Union Pond
Buckland
Oakland
PENN CENTRAL
PLATE NO. 2
CAHN ENGINEERS INC.
WALLINGFORD, CONNECTICUT
ARCHITECT-ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS
UNION POND DAM
HOCKANUM RIVER
MANCHESTER, CONNECTICUT

PHASE I INSPECTION REPORT

UNION POND DAM

SECTION I

PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of November 28, 1978 from Max B. Scheider, Colonel, Corps of Engineers. Contract No. DACW 33-79-C-0014 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

(1) Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

(2) Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

(3) To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

(1) Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

(2) A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

(3) Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

(4) An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features on the dam which need corrective action and/or further study.

1.2 Description of Project

a. Description of Dam and Appurtenances - The dam is a concrete gravity structure in an "L" shape. The total length of the dam is approximately 590 feet including the earth dike, with the left and right portions of the spillway being approximately 194 and 104 feet long, respectively. The top of the dam is approximately 33 feet above the bed of the Hockanum River. The spillway is a broad crested compound weir of trapezoidal cross section consisting of an outer concrete shell over an inner earth and rubble core. The existing dam was built over the original dam which was founded on a bedrock ridge. In 1972, No. 8 reinforcing bars 10 feet on center were grouted into 2 inch diameter holes drilled through the top of the present dam down 20 feet into the original dam. This was probably an attempt to increase the stability of the upper portion of the present dam.

The spillway crest is four feet below the top of the dam abutments. There are four outlets from the dam. There is a 42 inch low level outlet (invert elevation approximately 117.7) at the right end of the spillway, which is referred to on the existing 1901 plan as the "old waste gate". This gate, termed a "mud gate" by the owner, was opened by use of jacks when the pond was lowered for repairs in 1972. At the extreme left end of the spillway, there are two intermediate level sluice gates through the dam, both of which outlet at approximate elevation 130.1. The left gate is operational while the right floor stand to the gate will not function. The outlets are approximately 2 feet by 3 feet in size.

The fourth outlet is in the gatehouse at the extreme left end of the dam between the left dam abutment and the earth dike. The outlet feeds a cast iron conduit nine feet in diameter with an invert elevation of 127.5. The conduit runs under Union Street and flows back into the river further downstream. The gate to the conduit is presently inoperable, although the machinery is in good condition.

To the left of the gatehouse is the earth dike, which is approximately 175 feet long and has an average crest elevation of 146.7. A 6 inch thick wooden core wall consisting of three 2 inch planks was constructed along the centerline of the dike for a length unspecified on the existing plan.

b. Location The dam is located on the Hockanum River in a suburban area of the town of Manchester, County of Hartford, State of Connecticut. The dam is shown on the Manchester USGS Quadrangle Map having coordinates latitude N41° 48.0 and longitude W72° 31.7.

c. Size Classification - SMALL - The dam impounds 720 acre-feet of water (See Appendix Section D-7) with the pond level at the top of the dam, which at elevation 146.7, is approximately 33 feet above the level of the old streambed. According to the Recommended Guidelines, a dam with a height of less than 40 feet and a storage capacity of less than 1000 acre-feet is classified as small.

d. Hazard Classification - HIGH - Residential structures a minimum of 4 to 6 feet above the water level in the Hockanum River are located downstream of the dam. The closest structures are a house and garage approximately 8000 feet downstream near North Adams Street in Manchester. Also in this area are 12 commercial buildings, 5 residential structures and an apartment complex just downstream of North Adams Street.

e. Ownership - Town of Manchester
41 Center Street
Manchester, Connecticut
Mr. Jay Giles, Public Works Director
(203) 647-3142

f. Operator - None

g. Purpose of Dam - The dam was owned previously by the Cheney Brothers and the Connecticut Power Company. Present ownership by the Town of Manchester limits usage to recreational activities.

h. Design and Construction History - The following information is believed to be accurate based on the plans and correspondence available.

One of Connecticut's first paper mills was constructed on this site, but burned down in 1778, according to a sesquicentennial plaque on the side of the gatehouse dated 1823 to 1973. The date of the construction of the original dam is unknown. The dam was raised to its present height, and the gatehouse and 9 foot diameter conduit added in 1901 for the Cheney Brothers who owned it at that time. In 1972 repairs to the dam were carried out as described in

detail in the correspondence in Appendix Section B. Loose or deteriorated concrete on both upstream and downstream faces of the dam was jackhammered and removed. Voids in the dam which were discovered were filled by pressure grouting. Facing of the dam was done with wire mesh and gunite. Holes were drilled 20 feet deep from the top down into the lower portion of the present dam and into the old dam. Number 8 reinforcing bars were inserted and grouted, or pressure grouted if voids were discovered. Upon conducting the above work, it was discovered that the core of the dam was actually an earth and rubble core, rather than a solid concrete core. Subsequently, it was decided to seal the upstream face of the dam by excavating the fill adjacent to the dam and placing 3 inches of gunite over the face. Where this was not feasible, a clay blanket was placed adjacent to the dam extending away from the face up to 52 feet into the pond. Additional reinforcing of the dowsntream face was also recommended, as well as the installation of drilled weepholes near the downstream toe of the dam to provide pressure relief within the dam core. In addition, the controlled sluice gates were built and installed during these repairs to the dam.

Engineering for the above work was performed in part by Clarence Welti Associates, Inc., Macchi & Hoffman Engineers, Mr. Walter Senkow, Town Engineer of Manchester and Mr. William H.O'Brien III of the State Water Resources Commission.

i. Normal Operational Procedures - The single operational intermediate level sluice gate is opened in times of high water, or to control pollution from upstream sources, or when new construction requires the water level to be lowered. This was the case during our initial inspection when the water level was lowered for sewer and storm drain construction projects. When the pond was drained for the 1972 dam repairs, it was necessary to open the low level waste gate at the right end of the dam by means of special jacking equipment. To our knowledge, it has not been opened since that time.

1.3 Pertinent Data

a. Drainage Area - 53.9 square miles of rolling terrain. A large part of the drainage area is rural with scattered residential developments. A portion of the drainage area is made up of more heavily developed areas including Vernon and Rockville.

b. Discharge at Dam Site - Discharge from the pond is from 2 intermediate level sluices, a low level waste gate, and an inoperable 9 foot diameter conduit.

Outlet Works:	2 sluices-2'x3' @ el. 132 (approx.) 1 waste gate-42 inch dia. @ el. 117.7 9 foot dia. conduit @ el. 127.5
Maximum known flood at damsite:	21 inches over spillway
Ungated spillway capacity @ top of dam:	8400 cfs @ el. 146.7
Ungated spillway capacity @ test flood el.:	8400 cfs
Gated spillway capacity @ normal pool el.:	N/A
Gated spillway capacity @ test flood el.:	N/A
Total spillway capacity @ test flood el.:	8400 cfs
Total project discharge @ test flood el.:	N/A

c. Elevations - (Ft. above Mean Sea Level, U.S.G.S. Datum)

Streambed @ centerline of dam:	114 (approx.)
Maximum Tailwater:	N/A
Upstream inlet to 9 ft. conduit:	127.5
Normal pool:	142.7
Full flood control pool:	146.7
Spillway crest:	142.7
Design surcharge (Original Design):	N/A
Top of Dam:	146.7
Test flood design surcharge:	150.6±

d. Reservoir

Length of Max. pool:	3300+ ft.
Length of normal pool:	3300 ft. (approx).
Length of flood control pool:	N/A

e. Storage (See Appendix Section D-7)

Normal pool:	515 ac.-ft.
Flood control pool:	N/A
Spillway crest pool:	515 ac.-ft.
Top of dam:	720 ac.-ft.
Test flood pool:	N/A

f. Reservoir Surface

Top dam:	51.5+acres
Test flood pool:	N/A
Flood-control pool:	N/A
Normal pool	51.5 acres
Spillway crest	51.5 acres

g. Dam

Type:	Concrete gravity structure and earth dike
Length:	590 ft. (estimated from plans)
Height:	33 ft.
Top Width:	6 ft.
Side Slopes:	Dam - vertical upstream face Dike - 1.5H to 1V both slopes

Zoning:	N/A
Impervious Core:	N/A
Cutoff:	Ledge rock
Grout curtain:	N/A
Other:	Rubble interior of spillway

h. Diversion and Regulating Tunnel

Type:	Iron conduit, invert @ el. 127.5
Length:	370 ft. $\pm$ to outlet downstream
Closure:	N/A
Access:	Conduit buried in old canal
Regulating Facilities:	3 gates in gatehouse-inoperable

i. Spillway

Type:	Broad crested concrete weir of trapezoidal cross-section
Length of weir:	194 ft. (left section) 104 ft. (right section)
Crest el.:	142.7
Gates:	None
U/S Channel:	Clay blanket on shallow slope up to 50' into reservoir
D/S Channel:	Rock ledge and sand and gravel river bottom
General:	None

j. Regulating Outlets

Invert & Size:	2-2'x3' sluices @ el. 132 (approx.)

	1-42 inch dia. sluice @ el. 117.7
Description:	Sluices
Control Mechanism:	Intermediate sluices by 2 floor stands 42 inch by hand or jack operated mechanism
Other:	1 sluice gate inoperable

SECTION 2: ENGINEERING DATA

2.1 Design

a. Available Data - The available data consists of drawings, correspondence, calculations, and specifications by the owner, Clarence Welti Associates, Inc., Macchi and Hoffman Engineers, the State Water Resources Commission, and A.C. Rice, Engineer.

b. Design Features - With the exception of the 1901 plan, the existing data indicates the design features stated previously herein. The 1901 plan did not state that the core of the dam was rubble, or that the concrete spillway was actually only concrete facing.

c. Design Data - There ware no engineering values, assumptions, test results or calculations available for the original construction or the 1901 raising of the dam.

2.2 Construction

a. Available Data - There were no as-built plans or construction records available, with the exception of those pertaining to the 1972 repairs of the dam.

b. Construction Considerations - No information is available.

2.3 Operations

No formal operations records or data are known to exist.

2.4 Evaluation

a. Availability - Existing data was provided by the State Water Resources Commission. The owner made the dam accessible for visual inspection.

b. Adequacy - The limited amount of engineering data was adequate to perform only a very general stability analysis utilizing conservative assumptions. The actual condition and composition of the core of the dam are uncertain. The final assessment of this dam must be based primarily on visual inspection, past performance history, and hydraulic computations of spillway capacity based on approximate hydrologic judgement.

c. Validity - A comparison of record data and visual observations reveals no observable significant discrepencies in the record data.

SECTION 3: V[illegible] IN[illegible]

3.1 Findings

a. General - The general condition of the dam is poor. Inspection revealed areas requiring repair and maintenance, as well as some areas requiring further investigation.

b. Dam - The reservoir level was 13.2 feet below the top of the dam, at approximately elevation 133.5 on November 27, 1979 during our initial inspection. Upon subsequent inspections by Calvin Goldsmith on January 17, 22, and 26, 1979, the water level was approximately 0.5,6, and 8 inches over the spillway crest, respectively.

Crest - The crest of the dam is concrete with a gunite covering. The gunite was in good condition with minimal cracking. There are pipes at regular intervals along the crest of the spillway which, if struck by debris during heavy outflow, could contribute to localized instability of the top portion of the crest. It is this top portion of the crest that is of questonable stability already, as discussed in Section 6.

Upstream Face - The vertical upstream face exposed above the clay blanket is covered with gunite, which is in good condition with little or no cracking and only slight spalling.

Downstream Face - The downstream face of the dam is exposed down to the rock foundation. The dam shows considerable efflorescence and spalling as shown in Photos 1,3, and 4. Some cracks appear to be at least 2 to 3 feet deep. Seepage was observed through cracks in the dam, through the concrete-rock interface, through the exposed bedrock immediately downstream of the dam, and from weep holes near the toe of the spillway. At the time of the inspection, discharge from the seeps was small, however as the water level in the pond is raised, it is likely the amount of seepage will increase. (See Photos 4 and 6). The bedrock exposures are arkose sandstone with near-horizontal bedding.

The earth dike to the left of the gatehouse is in good condition. Both upstream and downstream slopes are grass-covered with evidence of minor erosion and sloughing only on the upstream face. There are trees growing on the dike adjacent to the gatehouse.

c. Appurtenant Structures - The gatehouse at the left end of the dam is in very poor condition. The wall of the gatehouse facing the dam has been demolished, exposing the inoperative gate mechanism of the 9 foot diameter conduit. The exposed portion of the trash racks to the conduit are badly bent and corroded. The concrete retaining wall to the left of the gate house has a large crack running diagonally from the upper right corner down towards the lower corner. The upper portion of the wall is displaced in an upstream direction a maximum of approximately 4 inches.

Immediately to the right of the gatehouse there are two intermediate outlets through the left dam abutment. The gate valves located on the upstream face of the dam are opened by manually operated mechanisms on top of the abutment. The left gate is operational while the right gate is separated from the floor stand and hence, cannot be opened. See Photo 8. The downstream buttresses adjacent to the outlets are spalled and exhibit significant efflorescence.

The low level waste gate is located at the extreme right end of the dam. A one inch wire mesh screen protects the upstream inlet from trash entering. The wire fencing at each abutment designed to limit access to the dam crest, has been vandalized and no longer serves its purpose.

d. Reservoir Area - The shoreline surrounding the pond is partially wooded and generally developed with single family residences.

e. Downstream Channel - The downstream channel is largely undeveloped, steep-sided and wooded down to the initial impact area.

3.2 Evaluation

Based on our visual inspection, it was possible to assess the dam as being generally in poor condition. The following features were identified which could affect the future condition and/or stability of the dam.

1. The cracking and spalling of the downstream face of the dam could lead to a weakening of the dam and a decrease in resistance to sliding and/or overturning.

2. The seeps observed through the cracks in the dam and through its contacts with the foundation bedrock tend to accelerate deterioration of the dam when water freezes and expands in the cracks. This probably accounts for the relatively rapid deterioration of the downstream face of the dam since its repair in 1972.

3. The elevation of the crest of the dike near the corner of the fence surrounding the gatehouse is 0.5 to 1 foot lower than the top of the dam. Should the dam ever be overtopped, a concentrated flow would result in this area which would severely erode the dike.

4. The roots of the trees at the right end of the dike near the gatehouse *could provide* seepage paths which, especially in times of high water, could lead to deterioration of the earth dike by erosion.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Regulating Procedures

The single operable sluice gate is used to control flow and lower water levels in the pond when pollution from upstream sources becomes abnormally severe. The water level has also been lowered recently to facilitate the construction of sewer and storm drain projects in the area of the pond. Daily lake level readings are not taken.

4.2 Maintenance of Dam

As was described previously in Section 1.2 G, "Design and Construction History," repairs to the dam and gate structures were last performed on a major scale in 1972. Only minor maintenance to gates and fencing has been performed since then on an as-needed basis.

4.3 Maintenance of Operating Facilities

The only maintenance performed to the operating facilities is the removal of logs or other debris from the sluice gates, and the repair of the gate mechanisms as needed.

4.4 Description of Any Formal Warning System In Effect

No formal warning system is in effect. The dam is checked periodically for problems during storms or times of very high water.

4.5 Evaluation

Maintenance of the dam is poor and requires a great deal of improvement. Due to the inoperable condition of the gates to the 9 foot diameter conduit and the right sluice gate, the operational procedures are quite limited.

A formal program of operation and maintenance procedures should be implemented, to include documentation providing complete records for future reference. A formal warning system should be developed and implemented within the time frame indicated in Section 7.1c. Remedial operation and maintenance measures are presented in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

a. General - The dam is a high spillage-low storage type project with a drainage area in excess of 50 square miles.

b. Design Data - No computations could be found for the original dam construction or the 1901 construction of the present dam.

c. Experience Data - No information on serious problem situations arising at the dam has been found, and it does not appear the dam has been overtopped. The maximum known height of water over the spillway was during an ice storm about 5 years ago at which time a nearby resident of the area reported measuring 21 inches of water over the spillway crest.

d. Visual Observations - Trees in the downstream channel could partially hinder flow during very high water, but this would not be a problem as the downstream channel is quite large immediately below the dam. Debris being carried downstream by heavy flows could cause partial blockage of the channel where it passes under the Union Street bridge, or could actually cause damage or the collapse of the bridge.

e. Test flood Analysis - The test flood for this high hazard, small size dam is equivalent to one-half of the Probable Maximum Flood (PMF). Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges", dated March, 1978, peak inflow to the reservoir is 31,000 cfs (Appendix D-8); peak outflow (Test Flood) is 30,500 cfs with the dam overtopped 3.9 feet (Appendix D-13). Based upon our hydraulics computations, the spillway capacity is 8400 cfs, which is equivalent to approximately 28 percent of the Test Flood.

f. Dam Failure Analysis - Utilizing the April, 1978, "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam breaching would be 41,600 cubic feet per second. A breach of the dam would result in approximately 15 foot high waves, both immediately downstream of the dam and at the houses and commercial buildings in the initial impact area 8000 feet downstream of the dam near North Adams Street (Appendix D-17).

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations - Severe deterioration of the concrete downstream face of the dam due to observed seepage could quite possibly endanger the future safety and stability of the dam.

The damage to the gatehouse is extensive. The structure or portions of it could be subject to future collapse endangering anyone in it, such as as children living in the area that might use the dam and gatehouse as a playground.

b. Design and Construction Data - Other than the one plan dated 1901 for the construction of the present dam, no data pertaining to the construction of the dam was available. Substantial repairs were performed in 1972 as described in Section 1.2g. The repairs included removal and replacement of deteriorated concrete and filling the jackhammered areas with pressure grout near the base of the downstream face. During the removal of deteriorated concrete, it was possible to observe the composition of the core of the spillway section. It was described as follows in a letter from Mr. O'Brien to the Town of Manchester, dated October 12, 1971.

"The jack-hammering of deteriorated surface material as called for on the approved plans had revealed that instead of a solid concrete overflow section on top of the old masonry structure, it was merely a shell of concrete varying from a 6-inch thickness on the downstream side to somewhat more on the upstream, with a core of trap-rock aggregate. It appeared that most of the aggregate had absolutely no cement around it and had been just dumped in using the downstream shell and both the old masonry dam and the upstream wall as forms. There was a fair amount of earth (loam) and root structures within the core exposed at one point on the downstream face."

The 1971-1972 repairs included the installation of vertical or almost vertical, No. 8 reinforcing bars spaced 10 ft. on centers, from the top of the spillway section of the dam. The specifications required installation of the bars in 20 ft. deep, 2 in. diameter holes, with subsequent grouting, and in addition, the specifications stated that "if large voids are encountered, pressure grouting may be required."

The analyses made of the stability of the dam by others prior to the 1971-1972 repairs indicated a very low factor of safety against overturning and sliding under high water when making the assumption that there are horizontal surfaces through the dam across which there is only frictional resistance to movement (no cohesion or tensional resistance). This assumption was made because of the extensive horizontal cracks observed at the time. Because the dam is not of solid concrete, the stability action of the reinforcing bars is difficult to assess. The procedure described in the specifications for the installation of the bars does not ensure that these bars will not corrode in the zone where they are exposed to seepage flow in the uncemented "trap-rock aggregate." Thus, on the basis of available information, long-term reliance on the reinforcing bars for stability is not warranted.

The design and construction data available is not sufficient to perform an analyses of the overturning and sliding stability of the dam. Major considerations affecting stability which are not known include the location and character of the dam-rock interface both under the original dam and the 1901 dam.

6.1.c <u>Operating Records</u> - There are no records available concerning the development of spalling and cracking or other features which influence stability.

6.1.d <u>Post-Construction Changes</u> - There are no records of post-construction changes other than those of the repairs discussed in Section 6.1.b.

e. <u>Seismic Stability</u> - The dam is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition - Based upon the visual inspection and past performance, the dam appears to be in poor condition. The general stability of the dam is questionable. The earth dike to the left of the gatehouse is in good condition with no evidence of sloughing or erosion. Areas of concern of the dam include the heavy deterioration of the downstream face of the spillway and seepage eminating from these deteriorated portions. The overall stability of the dam relating to the condition of the reinforcing bars installed in 1972, the composition of the dam core, and the amount and path of seepage through the dam is also in question. The condition of the 9 foot diameter conduit and gate is unknown. There are other less critical areas requiring attention, as well.

Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, peak inflow to the reservoir is 31,000 cubic feet per second; peak outflow (Test Flood) is 30,500 cubic feet per second with the dam overtopped 3.9 feet. Based upon our hydraulics computations, the spillway capacity is 8400 cubic feet per second, which is equivalent to approximately 28 percent of the Test Flood.

b. Adequacy of Information - The information available is not plentiful enough, nor is it accurate enough, to permit an in-depth analysis of the stability of the dam. Therefore, this assessment of the stability of the dam must be based upon visual inspection, past performance of the dam, and only rough checks of past computations by others.

c. Urgency - It is recommended that the measures presented in Section 7.2 be implemented immediately upon the owner's receipt of this report. The measures presented in Section 7.3 should be implemented within 1 year of the owner's receipt of this report.

d. Need for Additional Information There is a need for additional information as recommended in Section 7.2.

7.2 Recommendations

1. Based upon the rough computations in Appendix D, the dam spillway capacity will be exceeded by the Test Flood. More sophisticated flood routing should be undertaken by hydrolgists/hydraulics engineers to refine the Test Flood figures. A study should be undertaken and recommendations made to increase the spillway capacity based upon the refined Test Flood figures. Recommendations should also be made to increase the capacity of the low level outlets.

2. A registered professional engineer qualified in dam inspection should investigate the stability of the dam. In particular, the engineer should consider the effects of:

a. The degree of corrosion of the reinforcing bars installed in 1972.

b. The build up of hydrostatic pressure in the rubble core of the dam described as "trap rock aggregate", which appears to constitute the body of the dam.

c. The serious loss of structural quality and continuity of the downstream concrete shell which is the outer surface of the spillway face.

Subsequent recommendations should be made to satisfy the stability deficiencies of the dam, to eliminate seepage through the dam, and to provide methods of repair or replacement of the deteriorated surfaces of the concrete.

3. A registered professional engineer qualified in dam inspection should also be retained to investigate the 9 foot diameter conduit. A determination should be made of whether or not the conduit has been sealed off and if it has been, where. According to the Town of Manchester, a contract is to be let out to install a sewer line in Union Street, with an item included for the cutoff and sealing of the conduit. This will occur at least 80 feet from the gate structure, which is far enough so that deterioration of the gate structure and the remaining conduit could cause serious erosion of the dike, the left abutment of the dam, the left bridge abutment to the Union Street bridge, or Union Street itself. If the conduit is to be sealed, it should be sealed permanently as close to the gate as possible.

However, if possible, consideration should first be given to rennovating the conduit for use as another low level outlet during times of high water, or to lower the water level quickly should an emergency situation arise.

7.3 Remedial Measures

a. Operation and Maintenance Procedures - The following measures should be undertaken within the time frame indicated in Section 7.1.C, and continued on a regular basis where applicable.

1. Round-the-clock surveillance should be provided by the owner during periods of unusually heavy precipitation. The owner should develop a formal warning system for alerting downstream residents in case of an emergency.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.

3. A program of inspection by a registered, professional engineer qualified in dam inspection should be instituted on an annual basis. The inspections should be technical in nature, and should include the operation of the outlet works.

4. The badly damaged gatehouse is a hazard and should be made completely inaccessible to trespassing, or it should be removed.

5. The right sluice gate should be made operable, and the low level waste gate at the right end of the dam should be maintained regularly to render it easily operable.

6. The low areas of the earthen dike, particularly adjacent to the fence around the gatehouse, should be raised to the same elevation as the top of the dam.

7. Trees growing on the earthen dike near the gatehouse should be removed.

8. Contact seeps at the right dam abutment and along the toe of the dam-bedrock interface should be monitored regularly for significant increases in seepage volume not related to fluctuations of the pond water level.

9. The vertical pipes along the crest of the spillway (Photos 2 and 3) should be removed.

7.4 Alternatives

This study has identified no alternatives to the above recommendations and remedial measures.

APPENDIX

SECTION A: VISUAL OBSERVATIONS

VISUAL INSPECTION CHECK LIST

PARTY ORGANIZATION

PROJECT [illegible] POND DAM

DATE: NOV. 21, 1978

TIME: 8:30 AM

WEATHER: CLDY. COLD

W.S. ELEV. 335 U.S. ______ DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. PETER HEYMAN	PMH	CAHN ENGRS, INC.
2. CALVIN GOLDSMITH	CRG	CAHN ENGRS, INC.
3. GONZALO CASTRO	GC	GEOTECHNICAL ENGRS, INC.
4.		
5.		
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. CONCRETE ABUTMENTS	PMH, CRG, GC	
2. EARTH DIKE EMBANKMENT	PMH, CRG, GC	
3. GATEHOUSE AND 9' DIA. CONDUIT	PMH, CRG, GC	
4. LEFT ABUTMENT OUTLETS	PMH, CRG, GC	
5. SPILLWAY AND DISCHARGE CHANNEL	PMH, CRG, GC	
6.		
7.		
8.		
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

Page A-2

PROJECT [illegible] DATE NOV. 21, 1978

PROJECT FEATURE CONCRETE ABUTMENTS BY [illegible]

AREA EVALUATED		CONDITION
DAM EMBANKMENT		
Crest Elevation		EL. 146.7
Current Pool Elevation		EL. 133.5 (LATER [illegible] EL. 14[illegible])
Maximum Impoundment to Date		
Surface Cracks		NONE OBSERVED
Pavement Condition		SOME MINOR SURFACE SPALLING AND CRACKING
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		APPEARS GOOD
Horizontal Alignment		APPEARS GOOD
Condition at Abutment and at Concrete Structures		GOOD - SOME SPALLING OF CONCRETE ON TOP OF ABUTMENTS
Indications of Movement of Structural Items on Slopes		NA
Trespassing on Slopes		NA
Sloughing or Erosion of Slopes or Abutments		NA
Rock Slope Protection-Riprap Failures		NA
Unusual Movement or Cracking at or Near Toes		NONE OBSERVED
Unusual Embankment or Downstream Seepage		CONTACT SEEP AT RIGHT ABUTMENT
Piping or Boils		NONE OBSERVED
Foundation Drainage Features		NONE KNOWN
Toe Drains		NONE KNOWN
Instrumentation System		NONE

PERIODIC INSPECTION CHECK LIST

Page A-3

PROJECT [illegible] DAM DATE NOV 27, 1978

PROJECT FEATURE EARTH DIKE EMBANKMENT BY [illegible]

AREA EVALUATED		CONDITION
DIKE EMBANKMENT		
Crest Elevation		[illegible] - 146.7 (TYPICAL)
Current Pool Elevation		133.5
Maximum Impoundment to Date		
Surface Cracks		NONE OBSERVED
Pavement Condition		NA
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		TOO IRREGULAR TO JUDGE
Horizontal Alignment		TOO IRREGULAR TO JUDGE
Condition at Abutment and at Concrete Structures		EROSION ADJACENT TO GATE-HOUSE FENCE. HERE DIKE IS ≈1' BELOW TOP OF DAM ELEV.
Indications of Movement of Structural Items on Slopes		NA
Sloughing or Erosion of Slopes or Abutments		MINOR SLOUGHING OF U/S SLOPE
Rock Slope Protection-Riprap Failures		NO ROCK SLOPE PROTECTION
Unusual Movement or Cracking at or Near Toes		NONE OBSERVED
Unusual Embankment or Downstream Seepage		NONE OBSERVED - NO WATER BEHIND DIKE AT FIRST VISIT; THEN WATER BEHIND DIKE AND SNOW ON GROUND
Piping or Boils		NONE
Foundation Drainage Features		NONE OBSERVED
Toe Drains		NONE OBSERVED
Instrumentation System		NONE
Trespassing on Slopes		MINOR

PERIODIC INSPECTION CHECK LIST

Page 1-4

PROJECT [illegible] DATE [illegible]

PROJECT FEATURE GATEHOUSE AND [illegible] CONDUIT BY [illegible]

AREA EVALUATED		CONDITION
OUTLET WORKS-CONTROL TOWER		
a) Concrete and Structural		
General Condition		VERY POOR - 1 WALL DEMOLISHED AND GATE MECHANISM EXPOSED
Condition of Joints		GOOD, EXCEPT LEFT WINGWALL WITH HORIZONTAL CRACK
Spalling		SOME AROUND TRASH RACKS
Visible Reinforcing		NONE OBSERVED
Rusting or Staining of Concrete		NONE OF CONCERN
Any Seepage or Efflorescence		NONE OBSERVED
Joint Alignment		4 INCH DISPLACEMENT ON LEFT WINGWALL CRACK
Unusual Seepage or Leaks in Gate Chamber		NONE OBSERVED
Cracks		WINGWALL AND GATEHOUSE WALL
Rusting or Corrosion of Steel		TRASH RACKS BENT AND HEAVILY CORRODED
b) Mechanical and Electrical		
Air Vents		NA
Float Wells		NA
Crane Hoist		NA
Elevator		NA
Hydraulic System		NA
Service Gates		GATE TO 7' CONDUIT NOT OPERABLE
Emergency Gates		NONE KNOWN
Lightning Protection System		NA
Emergency Power System		NA
Wiring and Lighting System		NA

PERIODIC INSPECTION CHECK LIST

Page A-5

PROJECT [illegible] DATE NOV. 27, 1978

PROJECT FEATURE LEFT ABUTMENT OUTLETS BY [illegible] GC

AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		
General Condition of Concrete		GOOD
Rust or Staining		MINOR
Spalling		SOME AROUND SLUICE GATE OUTLETS
Erosion or Cavitation		NA
Visible Reinforcing		NONE OBSERVED
Any [illegible] or Efflorescence		NONE OBSERVED
Condition at Joints		SOME CRACKING OF BUTTRESSES AT JOINTS
Drain Holes		NONE OBSERVED
Channel		
Loose Rock or Trees Overhanging Channel		SOME ON LEFT ABUTMENT-NOT OF CONCERN
Condition of Discharge Channel		GRAVEL STREAMBED
UPSTREAM GATES		LEFT GATE ONLY IS OPERABLE
WASTE GATE-LOW LEVEL OUTLET		GATE OPERABLE WITH JACK 1 INCH WIRE MESH SCREEN AS TRASH RACK
9 FOOT DIAMETER CONDUIT		CONDITION NOT KNOWN-NO FLOW FROM D/S END OF CONDUIT

PERIODIC INSPECTION CHECK LIST

Page A-6

PROJECT UNION POND DAM DATE NOV. 27, 1978

PROJECT FEATURE SPILLWAY AND DISCHARGE BY HHH, CEH, GC

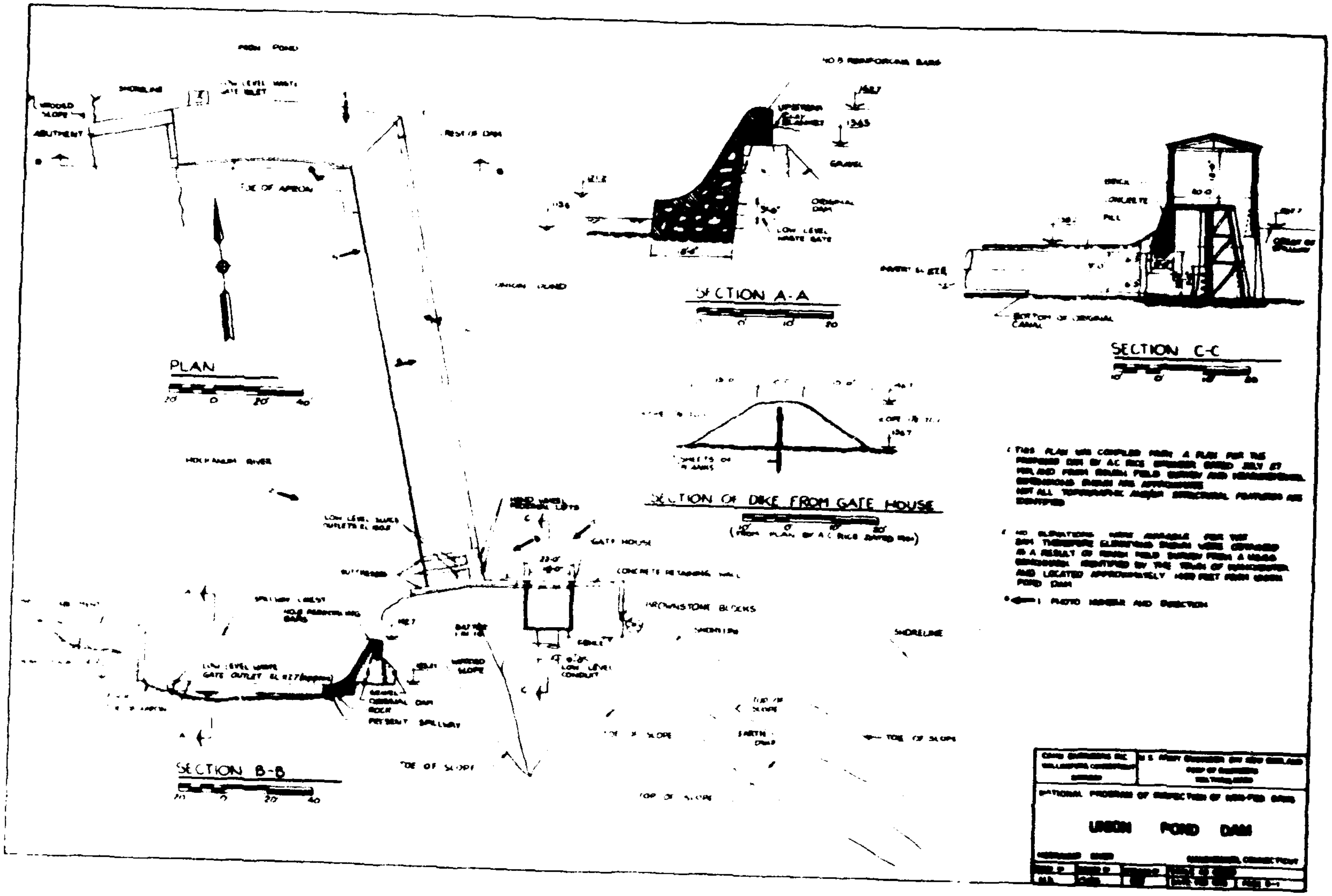
AREA EVALUATED		CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS		
a) Approach Channel		SHALLOW SLOPE - CLAY BLANKET
General Condition		GOOD
Loose Rock Overhanging Channel		NONE
Trees Overhanging Channel		NONE
Floor of Approach Channel		CLAY BLANKET
b) Weir ' Training Walls		
General Condition of Concrete		U/S FACE VERY POOR - CREST AND U/S FACE GUNNITE GOOD
Rust or Staining		D/S FACE STAINED
Spalling		D/S FACE HEAVILY SPALLED WITH CAVITATION AND CRACKING
Any Visible Reinforcing		NONE OBSERVED
Any Seepage of Efflorescence		D/S FACE SEEPAGE IS EXTENSIVE BUT NOT A LARGE FLOW
Drain Holes		SEEPAGE FROM NUMEROUS DRAIN HOLES
c) Discharge Channel		
General Condition		TREES AND LADEN WITH TRASH
Loose Rock Overhanging Channel		NONE OF CONCERN
Trees Overhanging Channel		ON RIGHT BANK AND IN CHANNEL
Floor of Channel		ROCK AND GRAVEL WITH TREES
Other Obstructions		BRIDGE COLUMNS AND SUPPORTS IMMEDIATELY DOWNSTREAM

APPENDIX

SECTION B: EXISTING DATA

APPENDIX

SECTION B. EXISTING DATA
UNION POND DAM



SHORELINE
ABUTMENT
TOE OF APRON
REST OF DAM
UNION POND
PLAN
20' 0 20' 40'
HOCKANUM RIVER
LOW LEVEL SLUICE OUTLETS
BUTTRESSES
SPILLWAY CREST
GATE HOUSE
CONCRETE RETAINING WALL
BROWNSTONE BLOCKS
SHORELINE
LOW LEVEL CONDUIT
LOW LEVEL WASTE GATE OUTLET
RIPRAPPED SLOPE
ORIGINAL DAM
ROCK
PRESENT SPILLWAY
TOE OF SLOPE
TOP OF SLOPE
EARTH DIKE
SECTION B-B
20 0 20 40
NO 8 REINFORCING BARS
UPSTREAM CLAY BLANKET
GRAVEL
ORIGINAL DAM
LOW LEVEL WASTE GATE
SECTION A-A
10 0 10 20
BRICK
CONCRETE
FILL
BOTTOM OF ORIGINAL CANAL
SECTION C-C
10 0 10 20
SHEETS OF PLANKS
SECTION OF DIKE FROM GATE HOUSE
(FROM PLAN BY A.C. RICE DATED 1901)
PHOTO NUMBER AND DIRECTION
SHORELINE
NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS
UNION POND DAM
MANCHESTER, CONNECTICUT

LIST OF EXISTING PLANS

"Gate House and Dam"
July 27, 1901
A.C. Rice, Engineer

"Typical Sectional View
Rehabilitation of Union Pond Dam"
September 23, 1970
Walter J. Senkow, Manchester Town Engineer

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
No date	Files	State board for the Supervision of Dams	Inventory Data	B-6
Sept. 5 1968	William D. O'Neill Director of Public Works Town of Manchester	William H. O'Brien III Civil Engineer	Inspection Report on Union Pond Dam	B-7
Aug. 10 1979	William D. O'Neill	Clarence Welti Associates Inc.	Inspection Report and repair recommendations	B-9
Sept. 24 1970	Contract Bidders	Town of Manchester Public Works Dept. Engineering Division	"Special Conditions Rehabiliatation of Union Pond Dam" with drawing	B-1[illegible]
Oct. 22 1970	William H. O'Brien III Water Resources Comm.	H. R. Hoffman P.E. Macchi & Hoffman, Engineers	Further recommendations for rehabilitation of dam.	B-14
Oct. 12 1971	Robert B. Weiss General Manager Town of Manchester	Wiliam H. O'Brien III	Necessity of revising dam rehabilitation plans due to discovery that 1901 plan of dam is incorrect.	B-16

DATE	TO	FROM	SUBJECT	PAGE
Oct. 14	Clarence Welti	Walter J. Senk w Town Engineer, Manchester	Analysis of Union Pond Dam for stability against overturning	B-18
Oct. 20 1971	Walter J. Senkow	William H. O'Brien III	Field inspection of construction work	B-21
Oct. 26 1971	Clarence Welti	Walter J. Senkow	"Analysis of Union Pond Dam-12 Foot Section"	B-23
Nov. 1 1971	Walter J. Senkow	Clarence Welti	Concurrence with Senkow's overturning calculations and supplemental recom-mendations (with sketch)	B-26
Nov. 5 1971	Files	Macchi & Hoffman, Engineers	Calculations concerning overturning of dam	B-29
No date	Files	The Penetryn System Inc. Rehabilitation Contractor	"Typical Dam Repair" Detail sketches of repairs	B-37
July 21 1972	Jose H. Cosio Chief Engineer Macchi & Hoffman	Walter J. Senkow	Description and sketch of clay blanket to be placed along upstream face of dam	B-38

DATE	TO	FROM	SUBJECT	PAGE
Aug. 7 1972	William H. O'Brien III	A.J. Macchi Macchi & Hoffman, Engineers	Recommendation that conditonal approval be granted to fill reservoir	B-40

NOTE: Additional correspondence not included herein is available from the State of Connecticut and Town of Manchester files.

STATE BOARD FOR THE SUPERVISION OF DAMS
INVENTORY DATA

NAME OF DAM OR POND Union Pond

CODE NO. H9.9 — Long. 72-31-43

LOCATION OF STRUCTURE: Lat. 41-45-00

Town Manchester

Name of Stream Hockanum River

U.S.G.S. Quad. Manchester Long. ______ Lat. ______

OWNER: ~~Connecticut Light & Power~~

Address ~~Manchester~~ Town of Manchester

Telephone 41 Center St.

, Manchester

Pond Used For: [illegible]

Dimensions of Pond: Width ______ Length ______ Area ~~[illegible]~~ 51.5 acres

Depth of Water below Spillway Level (Downstream) 20' ±

Total Length of Dam 300 ± Length of Spillway 200 ±

Height of Abutments above Spillway 3' ±

Type of Spillway Construction Concrete and stone

Type of Dike Construction ______

Downstream Conditions ______

Summary of File Data ______

Remarks This is a major structure and while it appears sound, should be inspected by Board Member.

September 5, 1968

Mr. William D. O'Neill
Director of Public Works
Town of Manchester
Manchester, Connecticut 06040

Subj: Union Pond Dam
Manchester

Dear Mr. O'Neill:

On August 21, 1968, the undersigned inspected the subject dam at your request.

There were no indications that this structure was in a hazardous condition, but there has been a visible lack of maintenance for many years and the structure is in need of repair before serious structural deterioration sets in.

The Water Resources Commission has jurisdiction over all dams, " - - - which by breaking away or otherwise, might endanger life or property - - ", as explained in the enclosed copy of the General Statutes.

The concrete on the downstream face of the spillway and apron had spalled off to a depth of approximately one foot and several square feet in area at several places, and cement which was used to repair cracks has fallen out in places. The supports for the stems which raise and lower the gates have become detached from the upstream face of the dam, and are so rusted and bent that they appear permanently inoperable with the existing mechanism.

Because of these facts, and because this dam would cause damage in the event of failure, and because continued delay in the repair of this dam could lead to an ORDER from this Commission to repair or remove the structure, which expense would no doubt be much greater at that time, it would seem prudent for the town to have an engineering report made on this structure to determine what repairs should be made, and to schedule such repairs sometime within the next year or so.

We have written to the previous owner of the dam, Connecticut Power Company, inquiring if they have any plans or specifications on this structure.

May we hear from you as to your intentions in this matter for our records?

Very truly yours,

William H. O'Brien III
Civil Engineer

WHOIII:vhb

Enc.

CLARENCE WELTI ASSOCIATES, INC
[illegible] SYCAMORE STREET • GLASTONBURY, CONN. 06033 (203) 633 462[illegible]

CLARENCE W WELTI [illegible]

EDWARD J [illegible]

AUGUST [illegible], 197[illegible]

[illegible] WILLIAM O'NEILL, DIRECTOR
DEPARTMENT OF PUBLIC WORKS
[illegible] OF MANCHESTER
[illegible] BUILDING
[illegible]CHESTER, CONNECTICUT

[illegible] EXAMINATION [illegible]

[illegible]:

[illegible] OUR ON-SITE EXAMINATION [illegible] THE DAM [illegible] WEEK [illegible] VIEWED THE PLANS TO DETERMINE [illegible] DEFICIENCIES.

[illegible] GENERAL THE [illegible] TO THE DAM [illegible] BE [illegible] REMOVING OF [illegible] OF DISINTEGRATED AND LOOSE CONCRETE AND GUNITING THE SPILLWAY FACE; THERE ARE [illegible] SOURCES OF APPREHENSION. AS AN EXAMPLE; IF IT WAS ASSUMED THAT EXTENSIVE LATERAL CRACKING EXISTED ACROSS THE TOP OF THE OLD [illegible] DAM AND OUT TO THE FACE OF THE SPILLWAY AND A FOOT OF WATER WAS ON THE SPILLWAY; STABILITY WOULD INDEED BE MARGINAL FOR THE TOP BLOCK OF CONCRETE.

SUPPLEMENTAL TO THE QUESTION OF STABILITY OF THE TOP CONCRETE IS THE DEPTH OF PRESENT DISINTEGRATION OF CONCRETE AND THE POSSIBLE EXISTENCE OF HIDDEN HORIZONTAL CRACKS. SUCH CRACKS AND DISINTEGRATION CAN WITH THE AID OF FROST PROMULGATE TO PRODUCE POSSIBLE CRITICAL STABILITY SITUATIONS. THE ONE SAFEGUARDING FEATURE AT THIS POINT IS THE GENERAL LACK OF VERTICAL CRACKING WHICH WOULD ISOLATE BLOCKS AND CAUSE DANGEROUSLY CRITICAL STABILITY SITUATIONS. THE NORTH EXPOSURE OF THE FACE OF THE DAM IS, HOWEVER, AN ACCELERATING FACTOR IN PROMULGATING FROST CRACKING AND DISINTEGRATION.

THE QUESTION OF FUNCTION ON THE GATES ADDS TO THE PROBLEM AT THE DAM. SHOULD THERE BE A CRITICAL SITUATION THESE SHOULD BE CAPABLE OF IMMEDIATE OPENING.

REVIEWING ALL OF THE ABOVE THE FOLLOWING ARE BASIC RECOMMENDATIONS:

1. OPEN EXISTING GATES TO ASCERTAIN FUNCTIONING OF GATES AND LOWER POND TO LEVEL AT OLD STONE DAM. GATES SHOULD BE REPAIRED.

[illegible handwriting]

2. Determine by probing the existence of possible jointing or cracking at the top of the old dam and the new spillway at the back of the dam.

3. Make jack hammer holes at 10' center along the top of the old stone dam to [illegible] foot depth to ascertain possible large voids in the stone dam.

4. Fill with pressure grout any voids found in item 4 and make additional holes if necessary.

5. On 10 foot center drill 2" holes through top of the spillway into the old stone dam and possibly on a slope into lower concrete for a depth of 20 feet. No.6 reinforcing bars should be placed in such holes and grouted. If large voids are encountered pressure grouting may be required.

6. On back and face of concrete dam chip out all cracks to full depth or a minimum of 1." ; remove all disintegrated concrete and probe large cracks with jack hammer to determine possibility of promulgation of such cracks.

7. After the above is completed the surface should be covered with 6" x 6" mesh, anchored into the concrete and a minimum of 4" of gunite concrete placed over the general area where cracking and disintegration are occurring. All deep, cleaned-out cracks and removed, disintegrated, portions should, of course, be first filled in with gunite concrete.

While the above represents a basic rehabilitation of the dam, it would certainly be necessary to carefully inspect the structure after drawdown to ascertain the necessity of the above repairs, as well as possible additional repairs. It is recognized that the analysis above is quite conservative. However, experience of the writer with dam failures indicate that causes of failure are often remotely elusive and yet the consequences of failure are almost always disastorous. It is therefore recommended that the water be maintained 6 feet below the top of the dam crest until damage, if any, is ascertained and repairs are made.

Very truly yours,

Clarence W. Welti, P.E.

TOWN OF MANCHESTER, CONNECTICUT
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

SPECIAL CONDITIONS

REHABILITATION OF UNION POND DAM

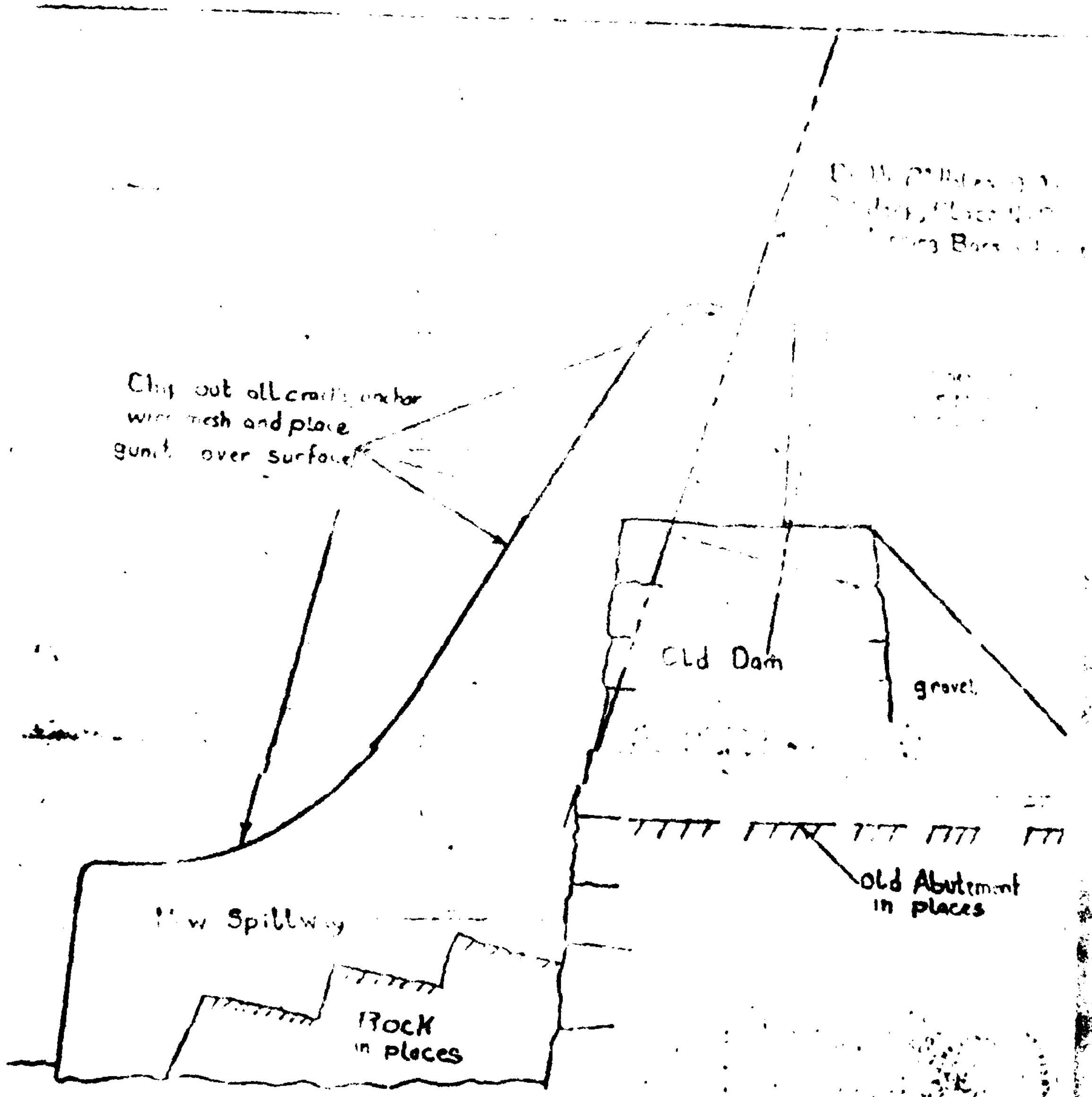
This work will consist of inspecting, evaluating, removing 6 inches to 12 inches of disintegrated and loose concrete and repairing with gunite. A field inspection of the Dam, by the prospective bidder, is encouraged and shop drawings for the repair of the sluice gates and waste gate are requested by the Town for its review. Waste gate repair will be an add item which the Town may or may not do. It is the Contractor's responsibility to seek out disintegrated concrete by probing, removing and repairing deteriorated or cracked areas with gunite. Special attention shall be given by probing for the existence of possible jointing or cracking at the top of the old Dam, new spillway at the back of the Dam, and the upstream face of the dam; this distance along the top of the Dam is about 300 feet. The Contractor will furnish all labor, tools equipment and materials for all work performed.

Contractor will make jack hammer holes at 10 foot centers along the top of the old stone Dam to a depth of 5 feet to determine any large voids in the stone Dam. These voids will be filled with pressure grout and additional holes will be made if necessary. Pressure grouting will be based on the contractor furnishing and placing 350 cubic feet of pressure grout. Should the amount of grout used increase or decrease from 350 cubic feet, the lump sum bid price will be increased or decreased based on this difference at a unit price of $10/cubic foot.

Contractor will drill 2 inch holes 10 feet on centers through the top of the spillway into the old stone Dam, possibly on a slope, into lower concrete, for a depth of 20 feet. Number 8 reinforcing bars shall be placed in such holes and grouted. If large voids are encountered, pressure grouting may be required. Pressure grouting will be based on the contractor furnishing and placing 350 cubic feet of pressure grout. Should the amount of grout used increase or decrease from 350 cubic feet, the lump sum bid price will be increased or decreased based on this difference at a unit price of $10/cubic foot.

Contractor on the back and face of the concrete Dam, the upstream face, and the downstream face, will chip out all cracks to full depth or a minimum of 12 inches. All disintegrated concrete will be removed and large cracks will be probed with a jack hammer to determine the possibility of the penetration of the cracks deeper into the Dam. Deficiencies will be repaired to the satisfaction of the Inspector. The surface will then be covered with 6 inch by 6 inch mesh, #12/12 gauge, anchored into the concrete, and a minimum of 4 inches of gunite concrete will be placed over the area where cracking and disintegration are occurring. All disintegrated portions shall be removed and cracks cleaned out and then filled with gunite concrete. The quantity of gunite claimed payment for will be verified by the inspector. Contractor will furnish all labor, tools and materials to perform the above work, and will be paid on a per cubic foot basis.

The Town will draw the water down to allow the Contractor to do the above work. Gunite shall be mixed in proportion of one sack of Portland Cement to 3-1/2 cubic feet of sand and mixed thoroughly in a dry state. Wire mesh will be 6 inch by 6 inch, #12/12 gauge and conform to the Standard Specification of the American Society for Testing Materials for "Cold Drawn Steel Wire for Concrete Reinforcement", Serial Designation A 82-34. The mesh shall be anchored to the existing concrete by 1/4 inch diameter expansion hook bolts 24 inches on center. Material shall not be placed on a frozen surface nor during freezing weather; below 32° Fahrenheit. Gunited surface will be sprayed with a liquid membrane curing compound. Curing compound shall be similar or equal to Denicon "Cure Hard" or Sealtight "Cure Hard" with fugitive dye and shall meet the latest A.S.T.M. Specification C-156.



Typical Sectional View scale 1"=40'
Rehabilitation of Union Pond Dam 9/23/70 W.J.S.

MACC[illegible] H[illegible] [illegible]AN • ENGINEERS
EXECUTIVE OFFICES • 4 GILLETT [illegible] • HARTFORD, CONN., 06105 • PHONE (203) 525-6631

A. J. MACCHI
H. R. HOFFMAN
J. J. SCHMID

ASSOCIATE CONSULTANT
PROF. C. W. DUNHAM

October 22, 1970

STATE WATER RESOURCES COMMISSION
RECEIVED
OCT 23 1970
ANSWERED ________
REFERRED ________
[illegible]D ________

State of Connecticut
Water Resources Commission
State Office Building
Hartford, Connecticut

Attention Mr. William H. O'Brien
Civil Engineer

Re: Union Pond Dam
Manchester, Connecticut

Gentlemen:

We have received and reviewed the following data as submitted by the commission, as requested in your letter of September 28, 1970.

1. Contract proposal for the rehabilitation of Union Pond Dam Bid No. B-27, as prepared by the Town of Manchester.

2. One copy of a typical sectional view of Union Pond Dam dated September 23, 1970 showing details of the dam rehabilitation.

3. One copy of a report dated August 10, 1970 prepared by Clarence Welti Associates Inc. for the Town of Manchester, Connecticut.

Field inspection trips were made to the dam site on Wednesday, October 14, 1970 and on Tuesday, October 20, 1970. The downstream face of the dam was inspected.

We are submitting a report of our inspections and findings on the safety of this structure as follows:

1. The dam, as it now stands, is in an unsafe condition.

2. We are in agreement with Clarence Welti Associates recommendation that a basic rehabilitation is necessary.

3. We are submitting the following recommendations and exceptions to the special conditions for the rehabilitation of

B-14

the dam as outlined in the specification as prepared by the Town of Manchester.

A. It would appear that the proposed method of repair does not fully insure against further deterioration because of the fact that at present it does not appear that any repair work is intended for the sources of leakage through the dam i.e. the upstream face of the dam.

It is recommended that the pond be drawn down to permit the entire upstream face of the dam to be repaired and made watertight.

B. All of the evaluation as to the extent of hidden cracking, voids or deterioration and to what extent they be repaired seems to be left with the contractor who has a fixed contract to do work that can only be determined after extensive probing. It is our opinion that fulltime inspection be done by a registered Professional Engineer.

C. We do not feel, at this time, that the rehabilitation of the dam, as presently outlined, will place the structure in what we regard as a safe condition.

Very truly yours,

MACCHI & HOFFMAN, ENGINEERS

H. R. HOFFMAN, P.E.

HRH/mcb

cc:

October 12, 1971

Town of Manchester
Municipal Building
41 Center Street
Manchester, Connecticut

Attention: Mr. Robert B. Weiss, General Manager

Re: Union Pond Dam
Manchester

Gentlemen:

On October 4, 1971, there was a field meeting at the subject dam with the undersigned, our consultant, Mr. Robert Hoffman and Mr. Walter Senkow of your engineering department in attendance.

The jack-hammering of deteriorated surface material as called for on the approved plans had revealed that instead of a solid concrete overflow section on top of the old masonry structure, it was merely a shell of concrete varying from a 6-inch thickness on the downstream side to somewhat more on the upstream, with a core of trap-rock aggregate. It appeared that most of the aggregate had absolutely no cement around it and had been just dumped in using the downstream shell and both the old masonry dam and the upstream wall as forms. There was a fair amount of earth (loam) and root structures within the core exposed at one point on the downstream face.

The design of repairs to the structure were based on plans of the dam dated 1901 which have now been shown to be incorrect.

We request that the original design engineer, Clarence Welti review the stability of this new-found section and revise the plans to ensure the continued safety of the structure. Such revisions must have prior approval of this Department before proceeding.

We understand that you are having the upstream face of the old masonry dam exposed by excavation. This is presumably to determine

the practicality or necessity of waterproofing the entire upstream face. It is most important to provide a positive seal against water entering the core.

Very truly yours,

William H. O'Brien III
Civil Engineer

WHO:mh

cc: Robert Hoffman
Walter Senkow
Commissioner Lufkin, Dept. of Environmental Protection

October 14, 1971

Clarence Welti Engineering Company
100 Sycamore
Glastonbury, Connecticut

Subject: Analysis of Union Pond Dam for Stability against Overturning

Dear Clarence:

I have enclosed calculations I intend to send to the State which will indicate that Union Pond Dam will not overturn, even if there was two feet of water going over its crest. I don't believe this has ever occurred. As we discussed the inner core was buoyed by water and weighed only 80#/cu. ft., hydrostatic pressure acted on the bottom of the Dam in a triangular pressure pattern.

I would be pleased to hear your comments on this matter.

Very truly yours,

Walter J. Senkow
Town Engineer

WJS:r
Enc.
cc: Robert B. Weiss, General Manager
William D. O'Neill, Director of Public Works

Analysis of Union Pond Dam for 10/5/77
Stability against being overturned

Total Area of Dam cross section, taking a section between the old and new dams. This section has a height of 8 feet on the upstream plane.

$A_1 = (\frac{1}{2})(3')(5') = 7.5^{\square'}$

$A_2 = (5')(2.5') + (\frac{2}{3})(2.5)(2.5) = 16.7^{\square'}$

$A_3 = (2')(7.25') \doteq 14.5^{\square'}$

$A_4 = (1')(\frac{8'+5.5'}{2}) = 6.3^{\square'}$

$A_5 = \quad = 3.3^{\square}$

$A_6 \quad = 2^{\square'}$

$A_7 \quad = 8^{\square'}$

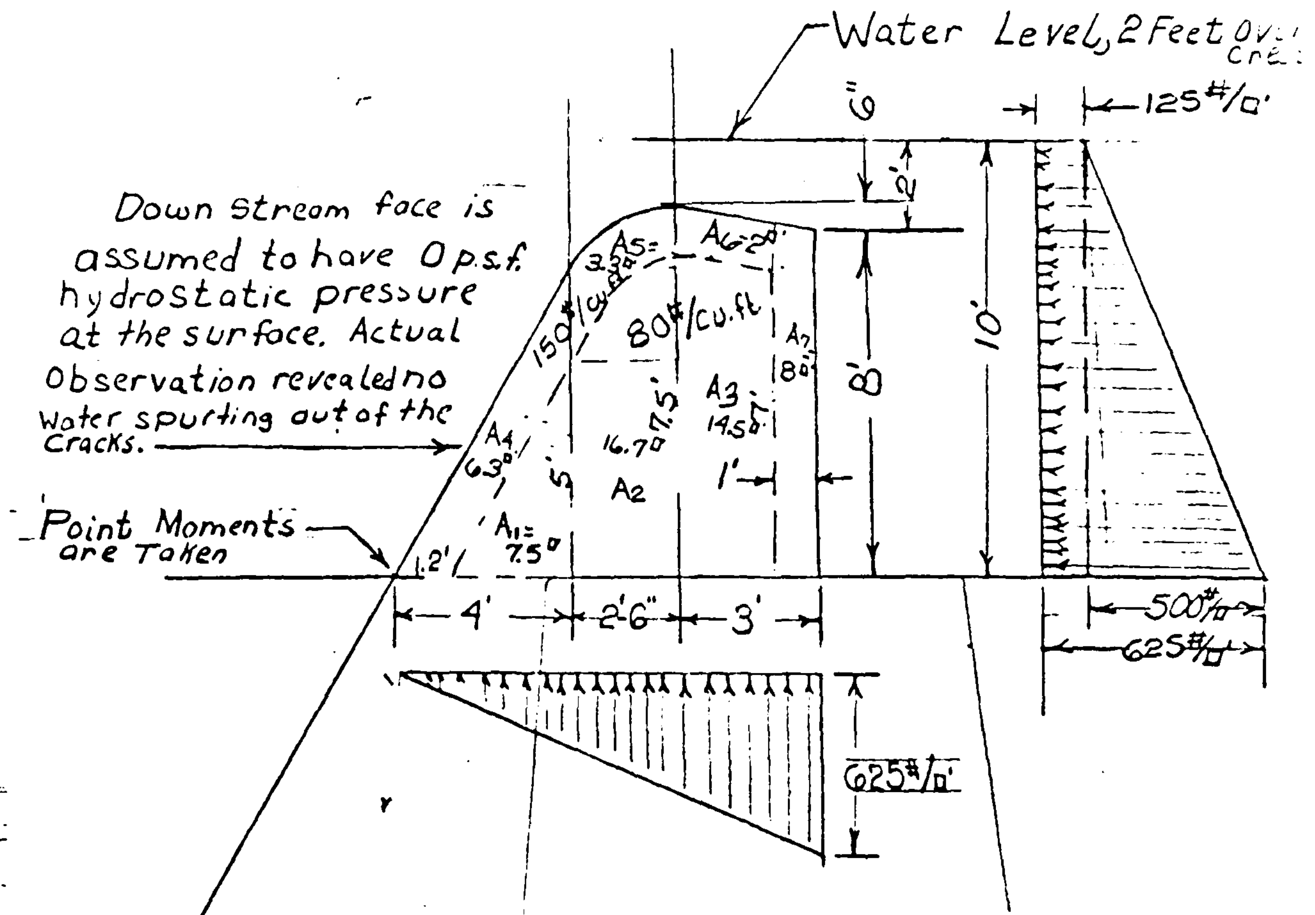
$\curvearrowright + \sum M = (7.5^{\square'})(1')(80^{\#}/cu.ft.)\{1.2' + (\frac{2}{3})(3')\} + (16.7^{\square'})(1')(80^{\#}/cu.ft.)(5.3') +$
$(14.5^{\square'})(1')(80^{\#}/cu.ft.)(7.5') + (6.3^{\square'})(1')(150^{\#}/cu.ft.)(2') + \ldots$
$(3.3^{\square'})(1')(150^{\#}/cu.ft.)(5.3') + (2^{\square'})(1^{\square'})(150^{\#}/cu.ft.)(7.5') + (8^{\square'})(1)(150)(9')$
$-(\frac{1}{2})(9.5')(1')(625^{\#}/\square)(\frac{2}{3} \times 9.5') - (125^{\#}/\square')(10')(1')(5') -$
$(\frac{1}{2})(500^{\#}/\square')(10')(1')(\frac{10'}{3})$

$= 1{,}920'^{\#} + 7{,}080'^{\#} + 8{,}700'^{\#} + 1{,}890'^{\#} + 2{,}624'^{\#} + 2{,}250'^{\#} + 10{,}800'^{\#}$
$-18{,}802'^{\#} - 6{,}250'^{\#} - 8{,}333'^{\#}$

$35{,}264'^{\#} - 33{,}385'^{\#} = +1{,}879'^{\#}$ against overturning

Factor of Safety $= \frac{35264'^{\#}}{33{,}385'^{\#}} = 1.06$

Dam will not overturn even if its inner core was completly saturated and buoyed up by a hydrostatic pressure acting underneath the dam. A theoretical 1 foot section was used. The actual moment against against overturning would still be greater because it is constructed in a L configuration. A 2 foot head was assumed over the top of the dam; this in actually has not been known to occur.



Old Dam

1' Outter shell of Concrete = 150#/cu.ft.

Inner Core is assumed pervious and is buoyed up by the water resulting in a net weight of 80#/cu.ft.

Section is assumed 1 foot thick

New Dam

Section A-B through Dam

Scale 1/4" = 1' Drawn by Walter Senko

10/13/71

Water Resources

October 20, 1971

Mr. Walter J. Senkow
Town Engineer
Town of Manchester
Municipal Building
41 Center Street
Manchester, Connecticut

Re: Union Pond Dam
Manchester

Dear Mr. Senkow:

At our field inspection of the subject dam on October 19, 1971, the following were in attendance: Robert Hoffman, our consultant; yourself; Mr. Anthony Fonte, contractor, of Penetryn Systems, Inc; and the undersigned.

Work was proceeding on removal of loose or deteriorated concrete on both faces of the dam. Some 4 or 5 feet of fill had been removed on the upstream side of the dam exposing portions of the shelf (shown on section C-D of the original plans about 8 feet below the spillway crest and a part of the "old dam"). In some areas the brownstone masonry could be seen and in other areas the original dam appears to have been covered by concrete - probably at an earlier time than the main concrete of the new dam because it was in extremely poor condition. In some areas the fill had been excavated, according to Mr. Senkow, to within a few inches of bed rock.

It was your intention to cover the upstream face with 3 inches of gunite to make this face waterproof. We would concur that this is an important step.

We wish to further emphasize that the stability of the structure under all conditions must be re-evaluated because the existing structure was found to be quite different from that assumed in the original analysis. The original design engineer should be called in at this point to re-evaluate the situation. We therefore request:

1. That Clarence Welti submit a letter to this department stating that he has inspected the existing conditions and has analyzed the stability of the structure, under certain defined conditions and stating his conclusions and recommendations.

2. That such conclusions and recommendations be submitted for the approval of this department.

Very truly yours,

William H. O'Brien, III
Civil Engineer

WHO:ljg

cc: William O'Neill
Robert B. Weiss
Robert Hoffman
Dan W. Lufkin, Commissioner
Department of Environmental Protection

October 26, 1971

Mr. Clarence Welti
100 Sycamore
Glastonbury, Connecticut

Subject: Analysis of Union Pond Dam - 12 Foot Section

Dear Clarence:

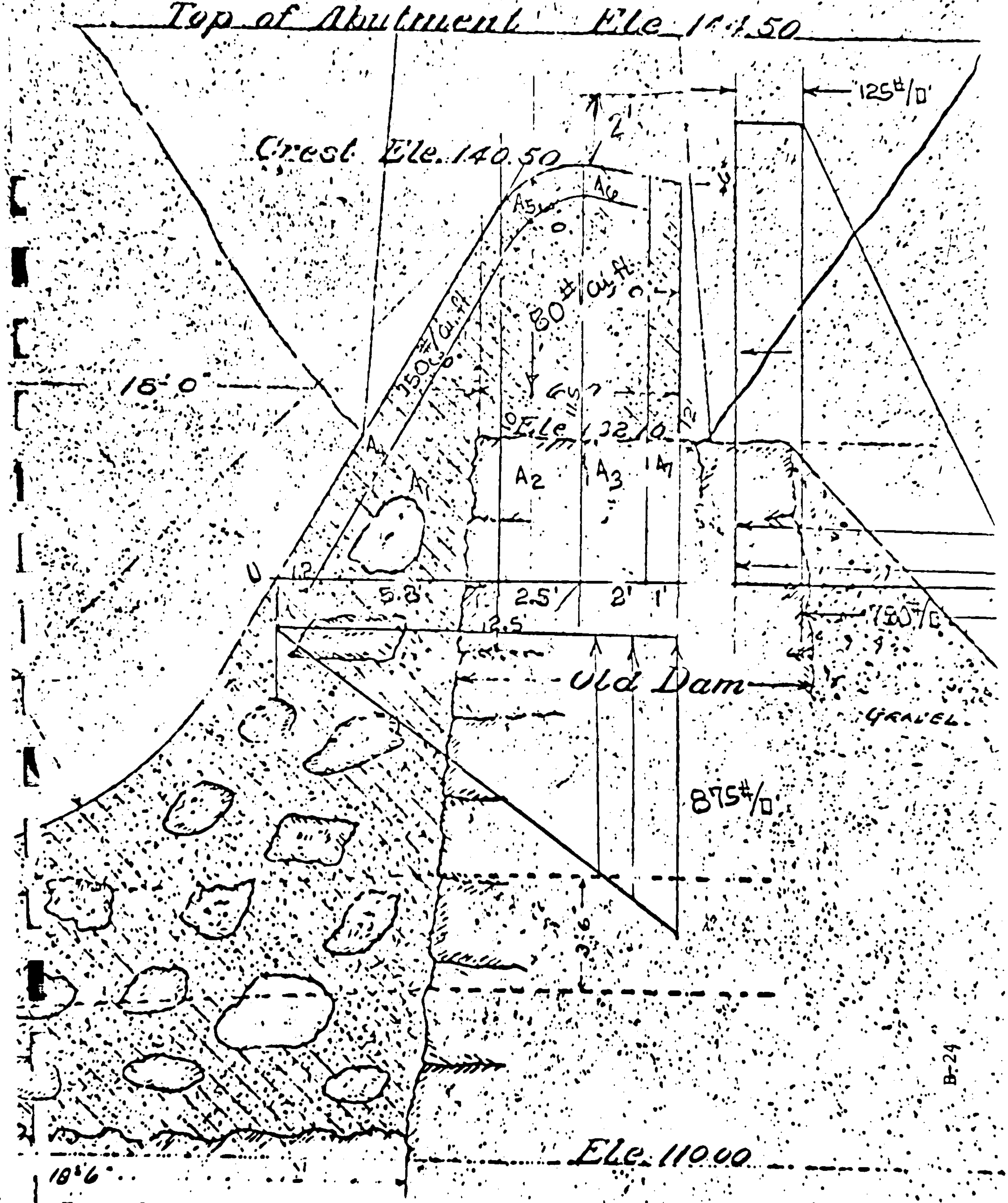
I have enclosed calculations showing a 12 foot upstream section with two feet of surcharge on it; again the dam won't overturn. The factor of safety appears to remain at a constant value, slightly greater than one along the dam's slope. Below this elevation the downstream face blends into a curve which creates a base far more stable than the above section. I'm also including a copy of Mr. O'Brien's letter to me dated October 20th.

Water Resources is requesting a letter that you approve of meshing and guniting the upstream face before starting the gunite operation. If you agree with the stability calculations please send them along saying you concur with it.

Very truly yours,

Walter J. Senkow
Town Engineer

WJS:r
Enc.
cc: William D. O'Neill, Director of Public Works



...al Elevation of Dam on "A-B"

2500 (62.5 × 14) = 875.0 #/□'
.25
875.0 #/□'

Section with 14 feet of Head - Bottom of Dam Using no old dam projection. A saturated buoyed inner Core was assumed; this would never occur to this extent in actuality. Dam would still not be overturned.

Area of Dam Section

$A_1 = (\frac{1}{2})(5.8')(10') = 29.0^{□'}$

$2 = (10')(2.5') + \frac{2}{3}(1.5)(2.5') = 27.5^{□'}$

$3 = (2')(11.25') = 22.5^{□'}$

$4 = (1')(\frac{13.5+11.5}{2}) = 12.5^{□'}$

$5 = 3.2^{□'}$

$6 = 2^{□}$

$7 = 12.2^{□'}$

$+\sum M_U = (80\#/cu.ft)\{(29^{□'})(1')(1.2' + (\frac{2}{3})(5.8')) + (27.5^{□'})(1')(8.3') + (22.5^{□'})(1')(\ldots$

$\ldots\#/cu.ft.)\{(2.5^{□'})(1')(3.5') + (3.2^{□'})(1')(8.3) + (2^{□'})(1')(10.5') + (12.2^{□'})(1')(\ldots$

$(875\#/□')(\frac{1}{2})(12.5')(1')(\frac{2}{3})(12.5') - (125\#/□')(14')(1')(7') - (150\#/ft)(\ldots$

$(\frac{14'}{3})$

$+\sum M_U = (80\#/cu.ft)(\overbrace{145 ft.^4 + 228 ft.^4 + 236 ft.^4}^{609 ft.^4}) + (150\#/cu.ft)(44 ft.^4 +$
$27 ft.^4 + 21 ft.^4 + 146 ft.^4) - 45,754'\# - 12,250'\# = 24,500'\#$

$= 48,720'\# + 35,700'\# - 82,504'\#$

$= 84,420'\# - 82,504 = 1,916'\#$

$F.S. = \frac{84,420'\#}{82,504'\#} = 1.02$ Safe Against over turning at dam bottom.

CLARENCE WELTI ASSOCIATES, INC.
100 SYCAMORE STREET • GLASTONBURY, CONN. 06033 (203) 633-4623

CLARENCE W. WELTI,
MANAGING ...

EDWARD J. P...
CHIEF ...

NOVEMBER 1, 1971

TOWN OF MANCHESTER
PUBLIC WORKS DEPARTMENT
MUNICIPAL BUILDING
MANCHESTER, CONN. 06040 ATT: MR. WALTER SENKOW

RE: UNION POND DAM

DEAR WALTER:

REGARDING THE ABOVE I HAVE REVIEWED YOUR ANALYSES RELATING TO OVERTURNING STABILITY AT 8' BELOW CREST AND 12' BELOW CREST. I HAVE ALSO VISITED THE DAM TO INSPECT SEEPAGE ON THE DOWNSTREAM SIDE AS WELL AS PORTIONS OF THE DAM ON THE DOWNSTREAM SIDE WHICH ARE BEING PREPARED FOR GROUTING.

AS PERTAINS TO THE STABILITY ANALYSES (OVERTURNING) THE APPROACH IS IN MY OPINION A CONSERVATIVE, RATIONAL APPROACH FOR THE FOLLOWING REASONS:

1. WHILE PORTIONS OF THE DAM INDICATE UNCEMENTED STONES, THE LARGE PORTION OF DAM DOES NOT INDICATE THIS CONDITION-TWO DIMENSIONAL ANALYSES PRESUMES UNIFORM LONGITUDINAL CONDITIONS.

2. WHILE THE DAM WAS WITHIN TWO FEET OF THE CREST, I INSPECTED THE DOWNSTREAM FACE AND ALL POSSIBLE SEEPAGE AREAS WERE AT PRACTICALLY ZERO HEAD, INDICATING NO DRAINAGE PATHS WHEREIN PRESSURE WAS NOT DISSAPATED ON THE UPSTREAM SIDE OR WITHIN THE DAM.

3. THE FACE OF THE DAM, EXCLUDING THE AREAS PRESENTLY EXCAVATED, WAS AND IS QUITE EVEN; INDICATING NO FROST HEAVING. SUCH A PHENOMENON WOULD HAVE TO OCCUR IF SUBSTANTIAL WATER WAS SEEPING TO THE DOWNSTREAM SIDE OF THE DAM.

<u>THE SAFETY FACTOR OF 1.03 TO 1.05 UNDER THE ABOVE CONDITIONS IS, IN MY OPINION, ADEQUATE</u>; SINCE NOT ONLY IS BUOYANT WEIGHT BEING USED FOR THE ENTIRE CORE (EXCLUDING L' SHELL), BUT TRIANGULAR WATER (WITH FULL HEEL) PRESSURE IS BEING USED. THIS CONDITION IN REALITY ASSUMES ALMOST FULL WATER PRESSURE ACROSS THE BASE OF THE SECTION OR NO WEIGHT OF THE WATER IN THE VERTICAL DIRECTION DOWNWARD

REGARDING SUPPLEMENTAL RECOMMENDATIONS THEY ARE AS FOLLOWS:

1. PLACE GUNITE OVER UPSTREAM SIDE TO ROCK WHERE POSSIBLE
2. WHERE NOT POSSIBLE CLAY BLANKET WILL BE PLACED. DEPTH AND DISTANCE WILL BE CALCULATED BY THE WRITER. THIS WOULD GREATLY DECREASE ANY PRESSURE HEAD AT SEEPAGE ZONE ON UPSTREAM SIDE.
3. INVESTIGATE WITH JACK HAMMER THE DEPTH TO SOUND CONCRETE AT DOWNSTREAM FACE. PLACE #6 HOOKED BARS IN GROUT HOLES AT LEAST 3 FEET INTO "SOUND" CONCRETE ON 2 FEET CENTERS.
4. HANG GRID OF #5 BARS AT 8" X 8" ON ABOVE #6 BARS PRIOR TO GROUTING.
5. AT BASE OF DAM WHERE "OOZING" IS OCCURRING EXCAVATE WITH JACK HAMMER TO EXAMINE SUPPLEMENTAL RECOMMENDATIONS.
6. AT 3 LOCATIONS IN DOWNSTREAM FACE PLACE DRAINS AS INDICATED ON THE ATTACHED. SINCE THESE AREAS WERE ONLY AREAS WHERE "BULGING" HAD OCCURRED IT IS PRESUMED THAT IF SEEPAGE OCCURS-ABOVE THE BASE-IT WILL OCCUR AT THESE AREAS.

WHEN YOU HAVE COMPLETED WORK ON ITEMS 2 & 5 PLEASE CALL ME.

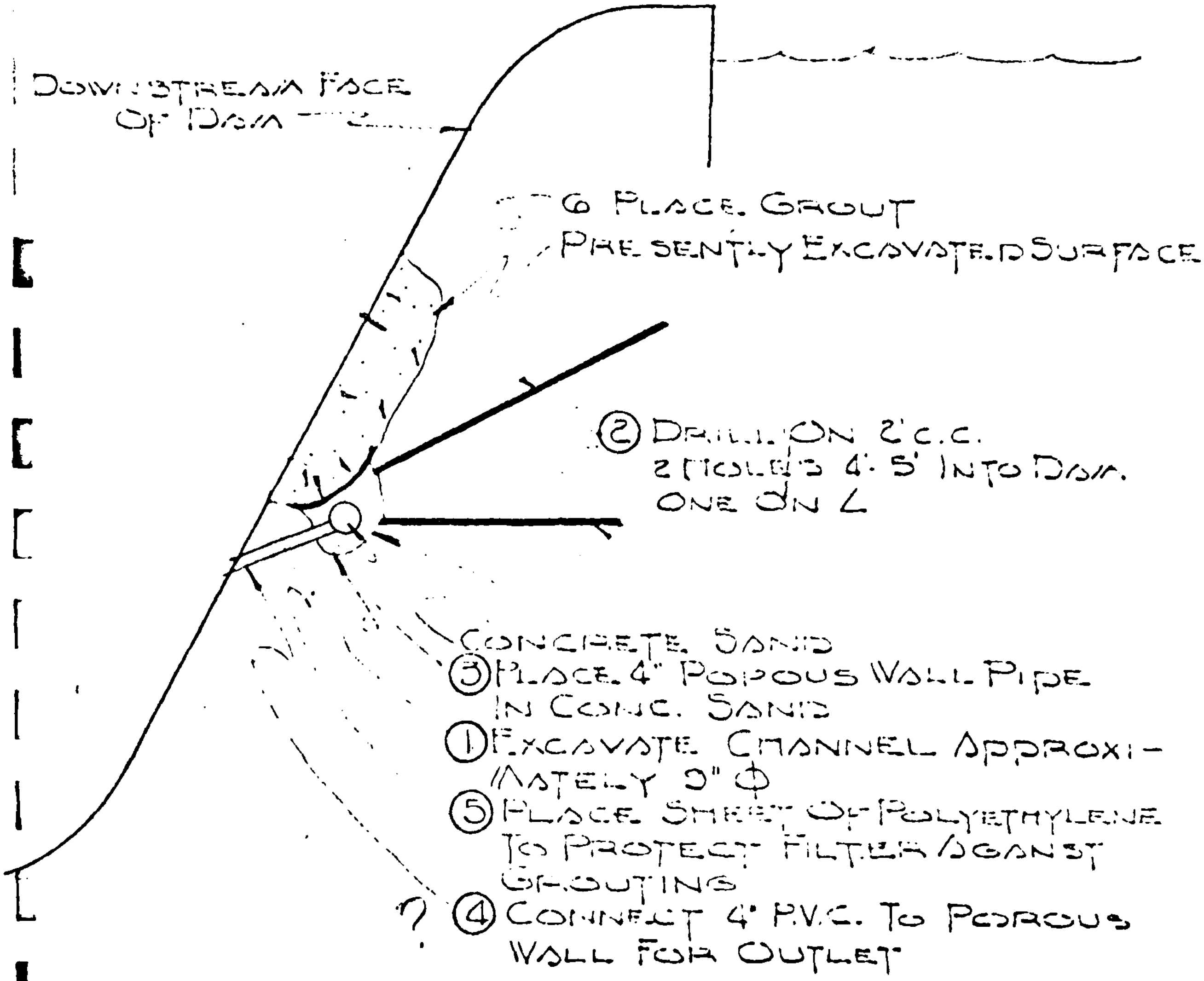
VERY TRULY YOURS,

CLARENCE WELTI ASSOCIATES, INC.

BY; Clarence Welti

CLARENCE WELTI, PH'D., P.E.

CWW:M
CC:FILE



C W A

RECOMMENDED SEEPAGE
RELIEF DRAIN-UNION POND DAM
MANCHESTER, CONN.

SCALE NONE		PROJ NO.
DR.		REV. DATE
APPROV.		DATE 11-1-71

CLARENCE WELTI ASSOCIATES
CLARENCE W. WELTI P.E.
GLASTONBURY, CONN 06033

Comments

(1) 80 lbs/ft³ for core material is not true as "Buoyancy" was considered separately. A figure of 110 lbs/ft³ or 120 lbs/ft³ is more in order.

(2) It is misleading to consider the overturning at the section under investigation. The real issue at the section is stress which is very low (± 40 lbs/□" for bending and 20 lbs/□" for shear).

(3) If as assumed that the portion under investigation is an independent entity, The factor of safety against overturing shown in the computation is low. A factor of safety of 2.0 is generally req'd.

(4) If the concrete of the section under consideration broken :

Total Sliding Force $= 0.125 \times 12 + \frac{1}{2} \cdot 0.75 \times 12 = 6.0^{K}$

Wt. of the Mass $= \overset{79}{(A_1 + A_2 + A_3)} \cdot 0.11 + \overset{29.2}{(A_4 + A_5 + A_6 + A_7)} \cdot 0.15$
$\quad - 0.0624\,\overset{108.2}{(A_1 + A_2 + A_3 + A_4 + A_5 + A_6 + A_7)}$

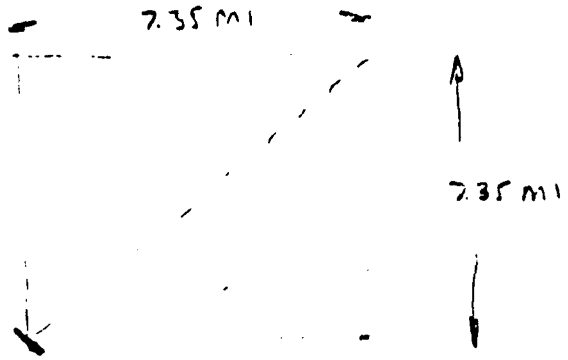
$= 8.7 + 4.4 - 6.8^{K} = 6.3^{K}$

With Friction factor say 0.6

$\therefore N \cdot f = 0.6 \times 6.3 = 3.8^{K} < 6.0^{K}$

Conclusion : If the concrete @ the section under considera: broken, the portion of the structure above that section is likely to be washed away.

SQ AREA →



A = 54.2 SQ MI

ACRES = 54.2 x 640 = 3480 ACRES

LENGTH OF TRAVEL

7.35 x 1.41 = 10.4 MI

x 5,280' = 54,000

POND EL. 146-00

HT 600'

$$\begin{array}{r} 600 \\ \underline{146} \\ 454 \end{array}$$

FT HIGEST POINT 454'

% SLOPE $\frac{500'}{54,000'}$ = .00925 %

SAY .01 %   CN 80

LAG 6 HOURS

SCS - TP-149 BULLETIN → $T_c = 1.67\,L$ = 1.67 x 6 = 10 HRS

intensity i = .5 IN/HR

Q = c i A

Q = .4 x .5 x 3,480 = 695 c.f.s.

100% RUNOFF Q = 1.0 x .5 x 3,480 = 1740 c.f.s.

CHECK CAPACITY OF SPILLWAY WITH 1' FOOT HEAD

$L = 300'$

$Q = 3.33\ L\ H^{3/2}$

$Q = 3.33 \times 300' \times 1^{1/2} \times 1^{2/2}$

$Q = 1{,}000 \times \sqrt{1}$

$Q = 1{,}000$ c.f.s.

CAPACITY WITH 2' HEAD

$Q = 3.33 \times 300 \times 2^{1/2} \times 2^{2/2}$ $3^{1/2}$.

$Q = 1{,}000 \times 2\sqrt{2} = 1{,}000 \times 2.82$

$Q = 2{,}820$ c.f.s. $>$ 1740 c.f.s.

100 YEAR STORM

CAPACITY WITH 3' HEAD = 5200 c.f.s

@ EAST HART. HOCKANUM RIVER

1,000 c.f.s. MEAN ANNUAL FLOW

745 SQ MI DRAINAGE

EFFECTIVE 57.5 SQ MI

FOR 100 YEAR STORM RATIO FACTOR = 3.7

Q = 1000 cfs x [illegible]

UNION POND DAM 10/20/70

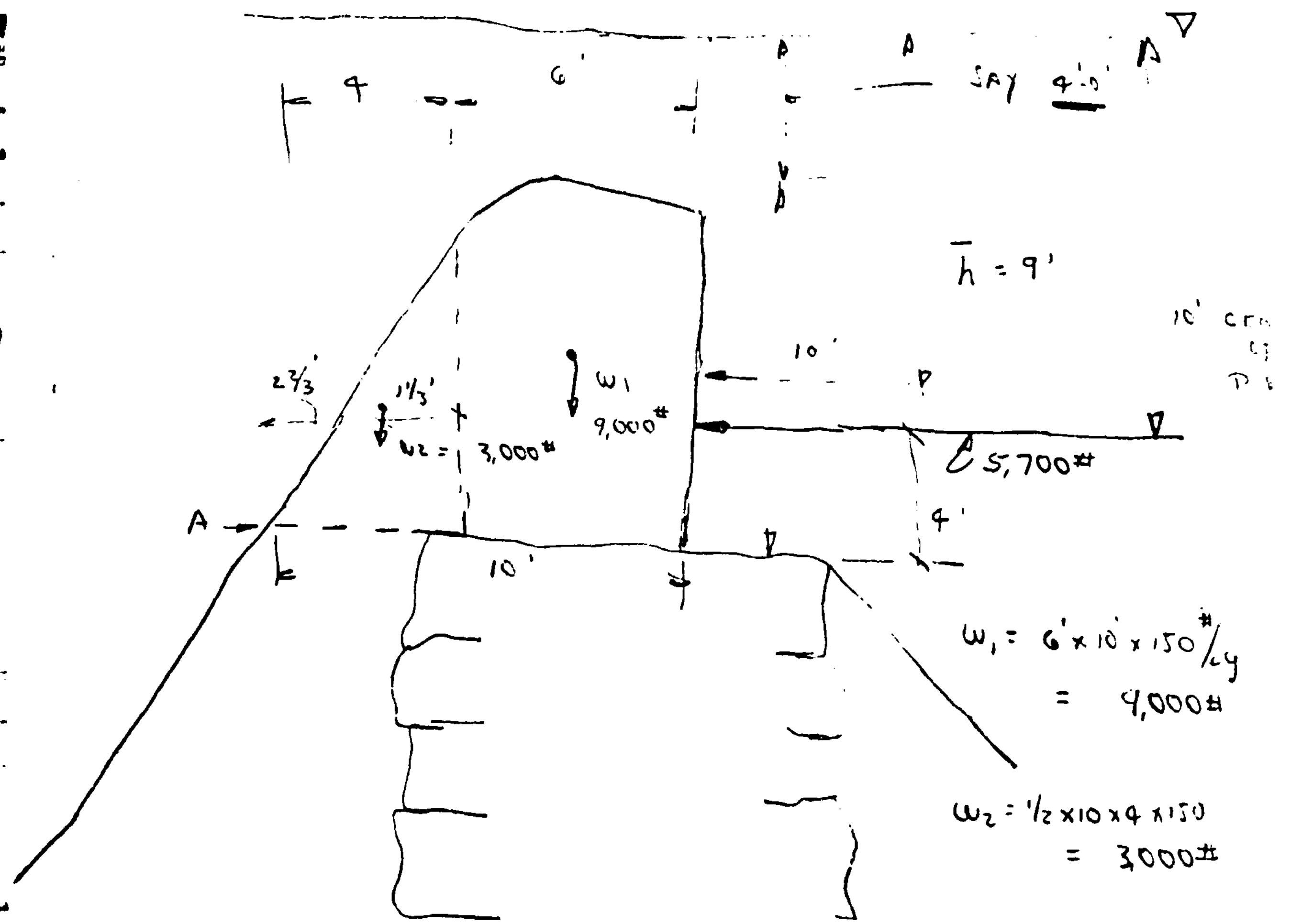
WATERSHED AREA

				SQ MI
A-1	.5 x 1.5	=	.75
A-2	3.75 x 4.25	=	16.00
A-3	1/2 x 2 x 4.25	=	4.25
A-4	1/2 x 1.75 x 1.25	=	1.10
A-5	1.75 x 5.5	=	9.60
A-6	2.5 x 3.0	=	7.50
A-7	3.0 x 2.25	=	6.75
A-8	1.75 x 1.25	=	2.20
A-9	1/2 x 3.0 x 3.0	=	4.50
A-10	1/2 x 1.5 x 2.0	=	1.50
				54.15 SQ MI

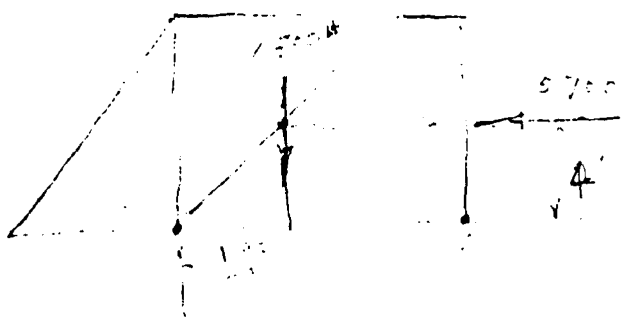
$P = A w \bar{h}$ $P = 10' \times 1' \times 62.4 \times 9' = 5{,}616\#$ SAY 5,700#

CENTER OF PRESSURE $y_{cp} = \dfrac{I_{cg}}{y_{cg} A} + y_{cg} = \dfrac{1 \times 10^3/12}{9 \times 10} + 9 = \dfrac{83.3}{90} + 9 = 9.9$

SAY 10'



CHECK OVERTURNING



$$\frac{[illegible]}{[illegible]} = \frac{[illegible]}{[illegible]}$$

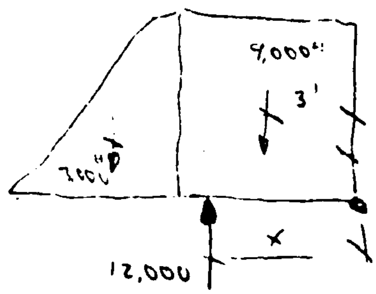
$$\frac{71800}{12000} = 6.00 \qquad e = 6 - 5.00 = 1.00$$

$e < \frac{10}{6}$ no tension

$\boxed{\text{no dowel reqd.}}$

See next page

TSW

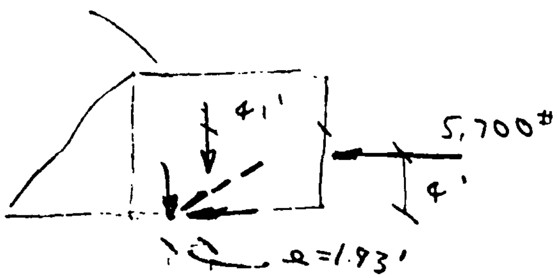


RESULTANT OF VERT

$$(12{,}000^{\#})(x) = 9{,}000 \times 3' + 3{,}000 \times 7\tfrac{1}{3}'$$

$$(12{,}000)(x) = 27{,}000 + 22{,}000$$

$$x = 49{,}000/12{,}000 = 4.08'$$



MID 3RD: $\frac{10}{6} = 1.67'$

CHECK RESULTANT IN MIDDLE 3RD

$$e = \Sigma M/V = \frac{5{,}700 \times 4}{12{,}000} = \frac{5{,}700}{3{,}000} = 1.93'$$

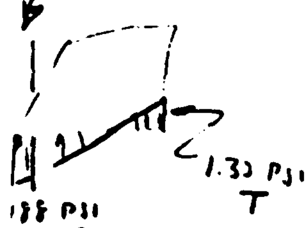
$$e > B/6$$

$$1.93 > 1.67$$ TENSION ON CONC

CHECK STRESS

$$f_{max/min} = \frac{\Sigma V}{BL}\left(1 \pm \frac{6e}{L}\right) = \frac{12{,}000^{\#}}{10' \times 1'}\left(1 \pm \frac{6 \times 1.93'}{10'}\right)$$

$$= 1200\ (1 \pm 1.16)$$

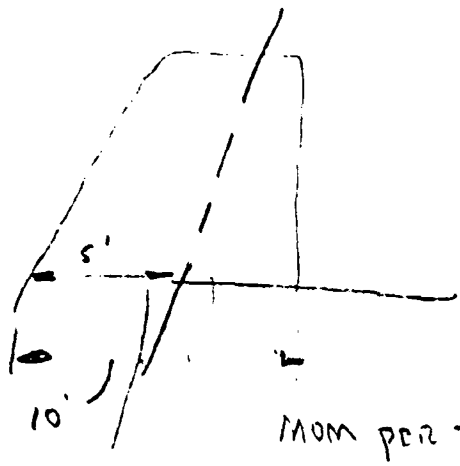


f_{comp} : 1200 psf × 2.16 = 2,700 psf. × 1/144 : 188 PSI

f_{TEN} : 1200 psf × −.16 = 192 psf. × 1/144 1.33 PSI

UNION POND DAM 10/20/70

CHECK #8 BAR AT 20'



MOM PER FT = 5,700# x 4' = 22,800 FT-#

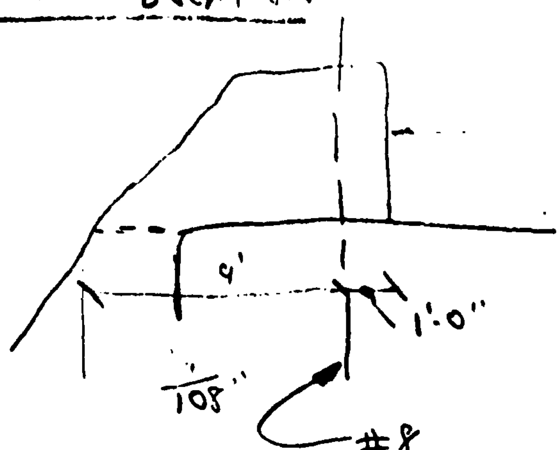
MOM PER 20' = 22.8'K x 20' = 456 'K

FORCE ON BAR = $\frac{456'K}{5'}$ = 910 K

BAR RESISTS .79 ☐" x 20.0K/IN² = 15.8 K N.G.

#8 @ 20' N.G.

BETTER LOCATION



$A_s = \frac{M}{1.44\,d} = \frac{22.8'K}{1.44 \times 108"}$

A_s = .147 ☐"/'

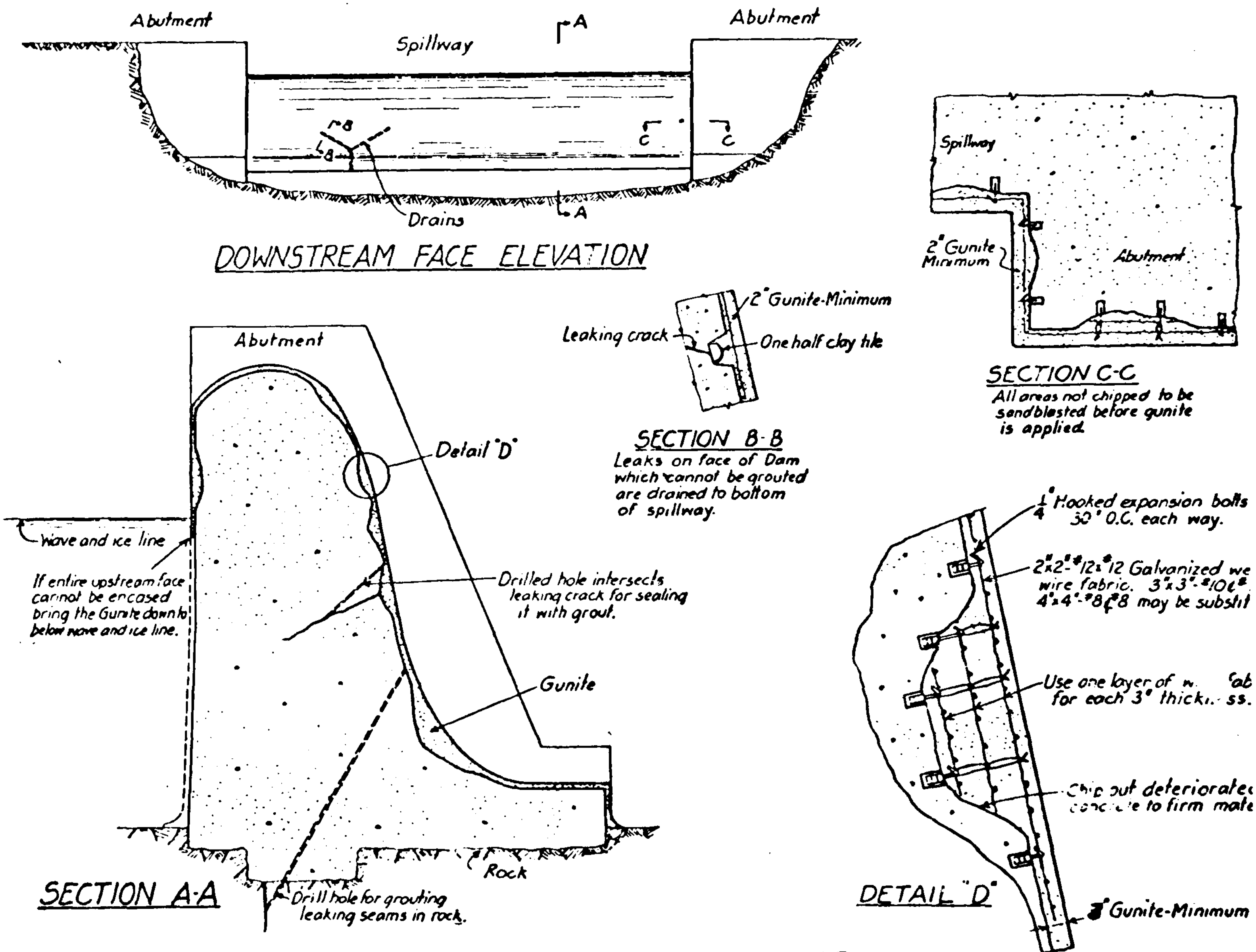
#8 A_s = .79 ☐"

SPACING = $\frac{.79☐"}{.147☐"/'}$ = 5.4'

SAY @ 5'-0" O.C

B-36

PENETRYN
P. O. BOX 146 EVERETT, MASSACHUSETTS 02149
617-389-1525



TYPICAL DAM REPAIR

Bell

July 21, 1972

Jose M. Cosio
Chief Engineer
44 Gillett Street
Hartford, Connecticut 06105
Macchie - Hoffman Engineering

Subject: Union Pond Dam - Clay Blanket

Dear Mr. Cosio:

As you requested after visiting the construction site, I am submitting a planned sketch of the clay blanket limit behind the dam. Also shown on this sketch is the depth of the clay blanket at the location. As you may recall the depth of the clay is greater than the hole that was dug. I am doing this as you requested at the site and in your letter of July 12.

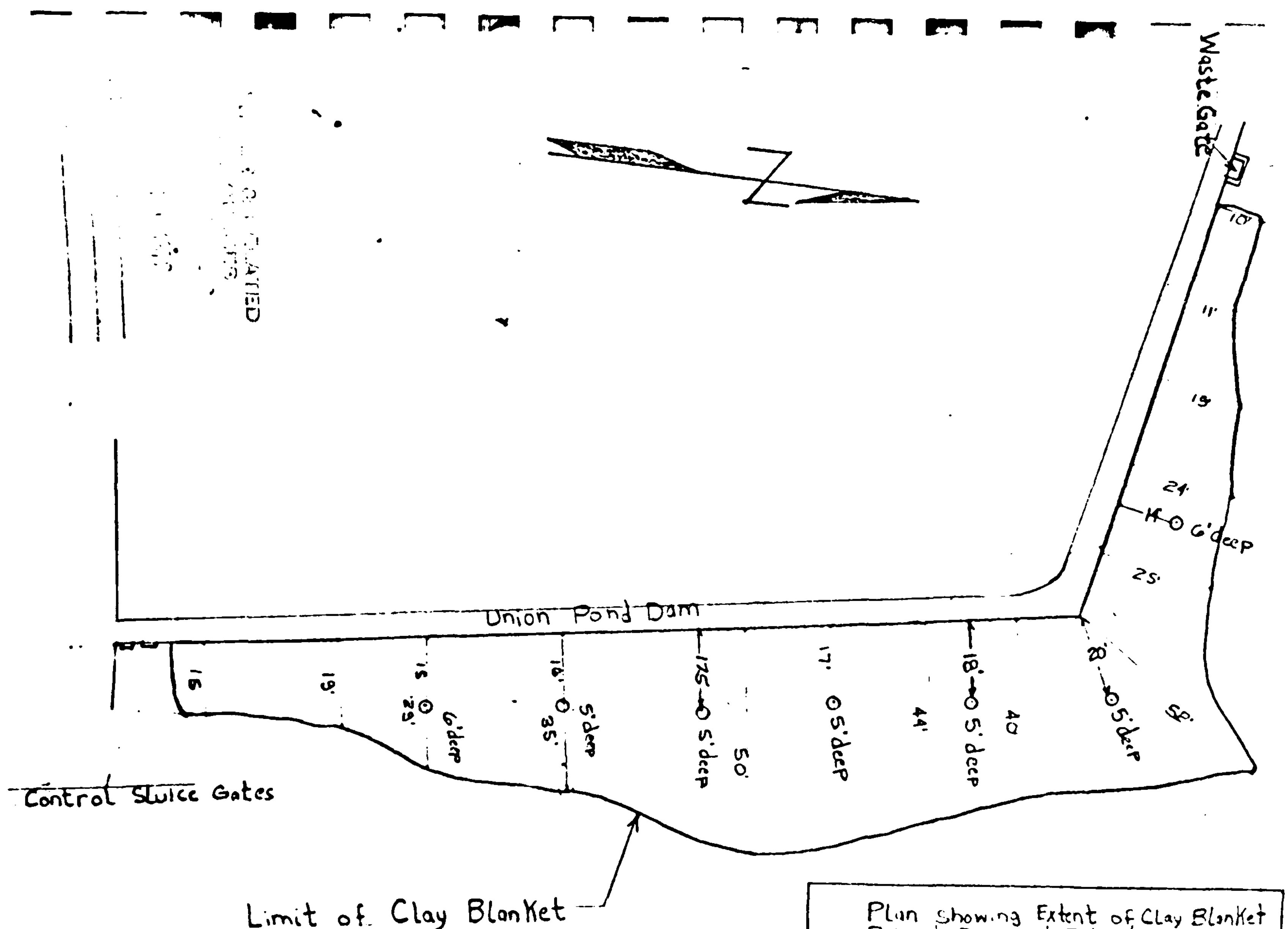
As a matter of general information I would like to mention that the controlled sluize gates have been built, mounted and are in operation. A wall has been built around the waste water gate. With this information I hope in your judgement, that you will recommend that the Department of Environmental Protection issue the Town of Manchester a certificate of approval.

If, however, in your opinion, there are other matters to be resolved, please let me know and I will do whatever I can.

Yours truly,

Walter J. Senkow
Walter J. Senkow
Town Engineer

WJS/dc
cc: William H. O'Brien III
William D. O'Neill
Robert Weiss



Plan Showing Extent of Clay Blanket
Behind Dam and Test hole Depths
Scale: 1" = 30' July 20, 1972
Walter Sankow

MACCHI & HOFFMAN • ENGINEER

EXECUTIVE OFFICES • 44 GILLETT STREET • HARTFORD, CONN., 06105 • PHONE (203) 525-6

A. J. MACCHI, P.E.
H. R. HOFFMAN, P.E.
MICHAEL GIRARD

ASSOCIATE CONSULTANT
PROF. C. W. DUNHAM

August 7, 1972

Dept. of Environmental Protection
Water & Related Resources
165 Capitol Avenue
Hartford, Connecticut

Attention Mr. William H. O'Brien III

Re: Union Pond Dam
Conditional Approval Recommended
Supersedes Our Letter 7/24/72

Gentlemen:

We have received the plan showing the extent of the clay blanket placed behind the recently repaired Union Pond dam, from the Town of Manchester.

As the clay blanket placed behind the dam is used to seal the upstream face of the dam itself, the extent of the blanket away from the dam is not critical due to the fact that the dam rests on a rock foundation, as indicated on the drawings.

Due to limitations in being able to verify the actual results obtained by grouting and actual condition of the original portion of this dam, it is recommended that conditional approval be granted at this time, to fill the reservoir behind the dam and after approximately six months, a reinspection be made to verify leakage through the dam.

Very truly yours,

MACCHI & HOFFMAN, ENGINEERS

A. J. MACCHI

WATER & RELATED RESOURCES
RECEIVED
AUG 1972
ANSWERED
REFERRED
FILED

B-40

APPENDIX

SECTION C: DETAIL PHOTOGRAPHS

PHOTO NO.1 - General view of downstream face of left spillway section and natural bedrock foundation.

PHOTO NO.2 - Downstream view of sluice gate outlets and gatehouse.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Union Pond Dam Hockanum River Manchester, Connecticut
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER		CE # 27 595 DATE Feb 1979 PAGE C-1

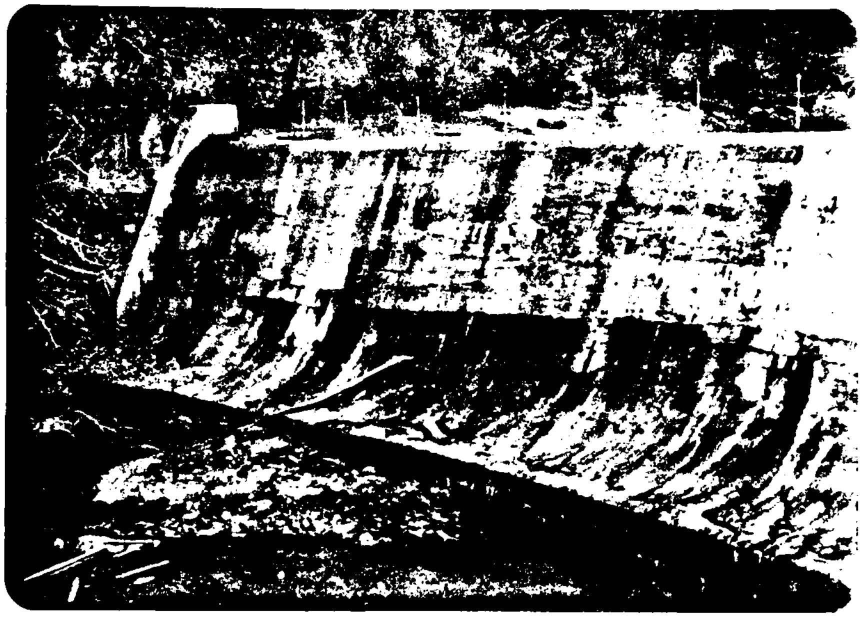


PHOTO NO.3 - General view of downstream face of right spillway section, right abutment, and low level waste gate outlet.

PHOTO NO.4 - Close-up of seepage and weepholes in downstream face of spillway.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Union Pond Dam Hockanum River Manchester, Connecticut CE # 27 595 DATE Feb 1979 PAGE C-2
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER		

PHOTO NO.7 - View of damaged gatehouse and bent, corroded trash racks.

PHOTO NO.8 - Upstream view of two (2) sluice gates. Only one is operable and open at time of inspection.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Union Pond Dam Hockanum River Manchester, Connecticut CE # 27 595
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER		DATE Feb 1979 PAGE C-4

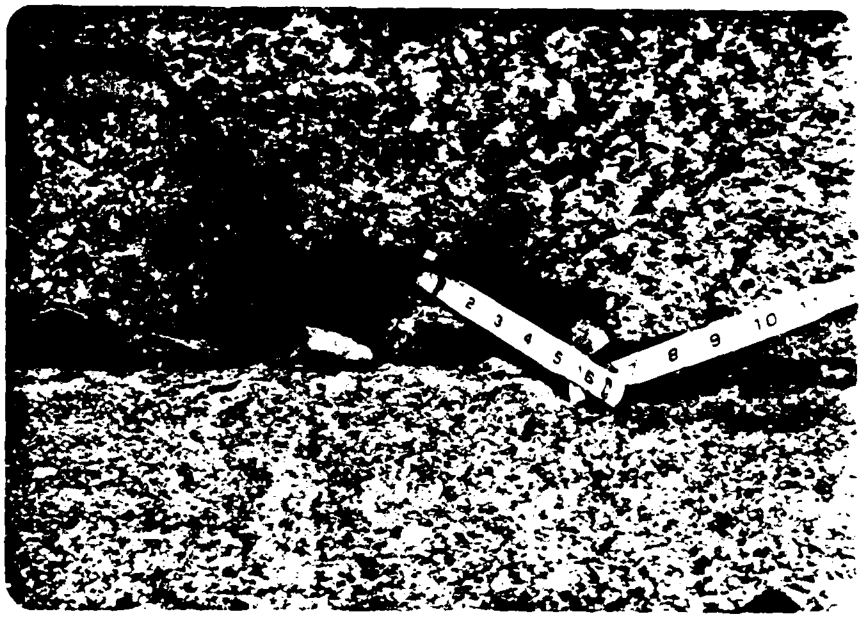


PHOTO NO.5 - Close-up of void in left spillway section downstream face. Note deteriorated gunnite facing.

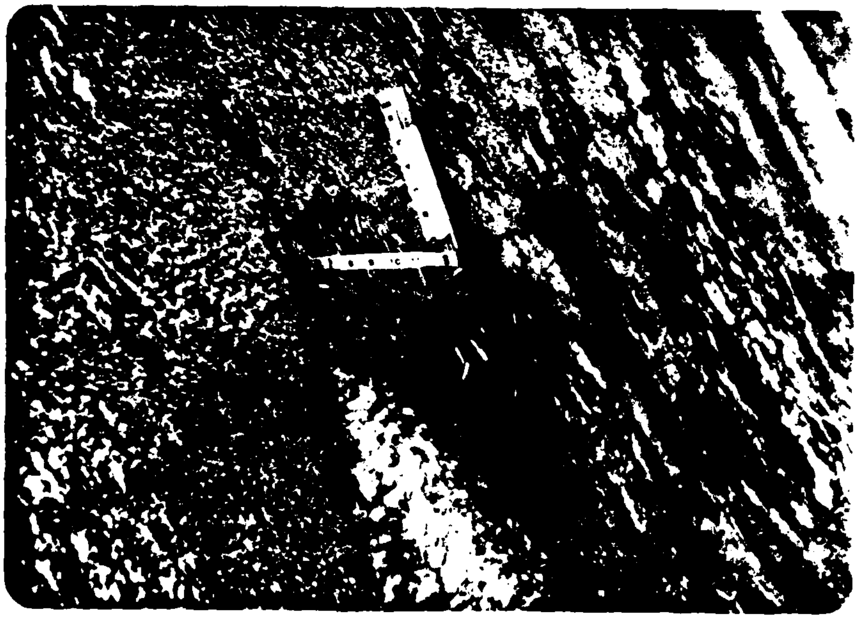


PHOTO NO.6 - Close-up of pressure relief weep hole drilled in downstream face of spillway.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Union Pond Dam Hockanum River Manchester, Connecticut
CAHN ENGINEERS INC. WALLINGFORD, CONN ARCHITECT —— ENGINEER		CE # 27 595 DATE Feb 1979 PAGE C-3

APPENDIX

SECTION D: HYDRAULIC/HYDROLOGIC COMPUTATIONS

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

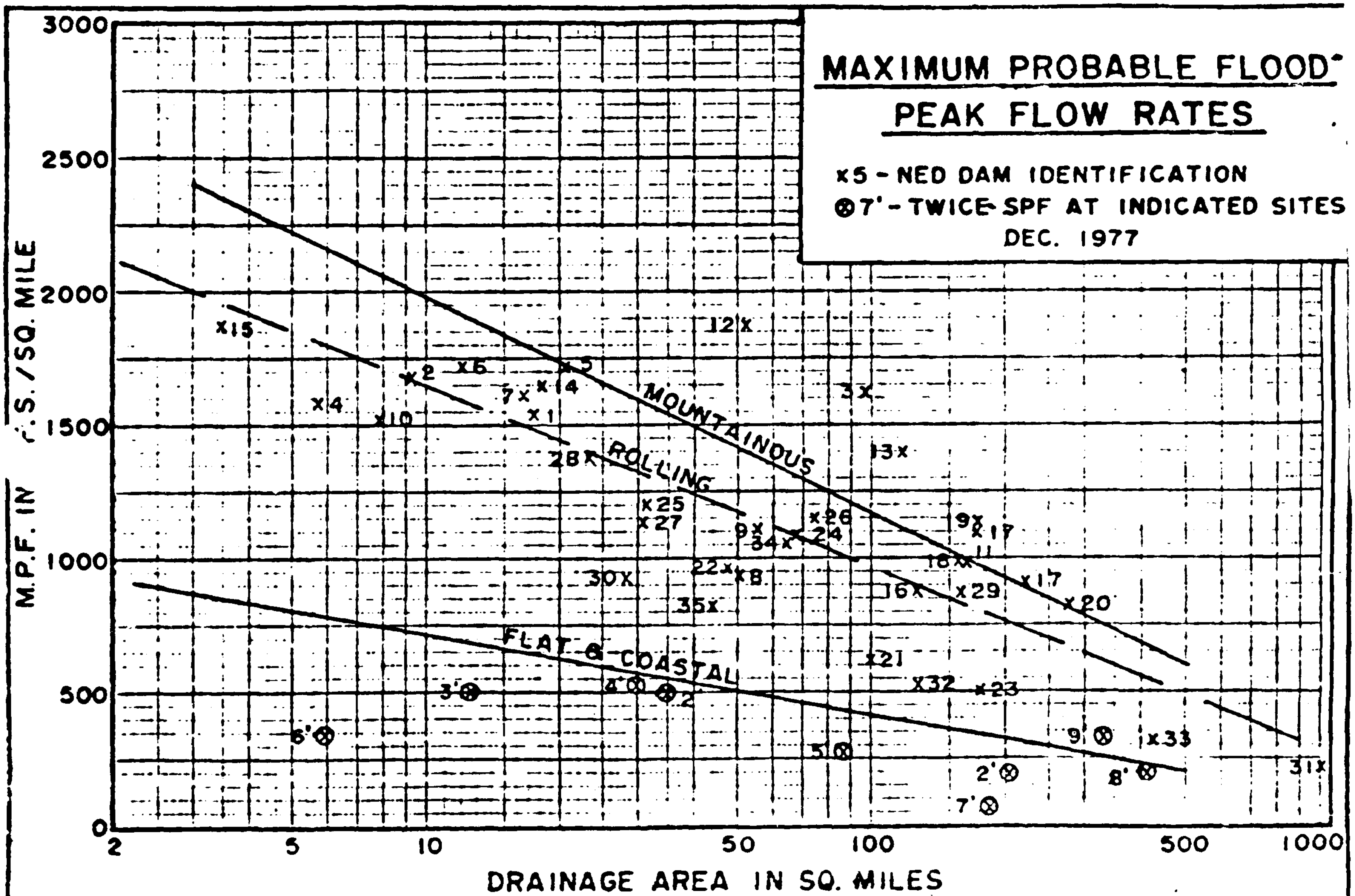
March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330

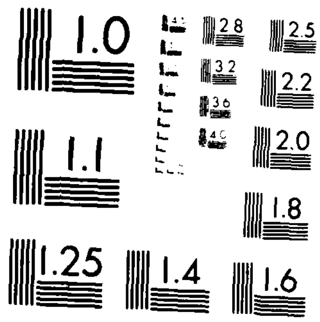

MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977
MOUNTAINOUS
ROLLING
FLAT & COASTAL
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000

AD-A142 621 NATIONAL DAM INSPECTION PROGRAM UNION POND DAM (CT 00013) UPPER CONNECTIC..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV FEB 79

2/2

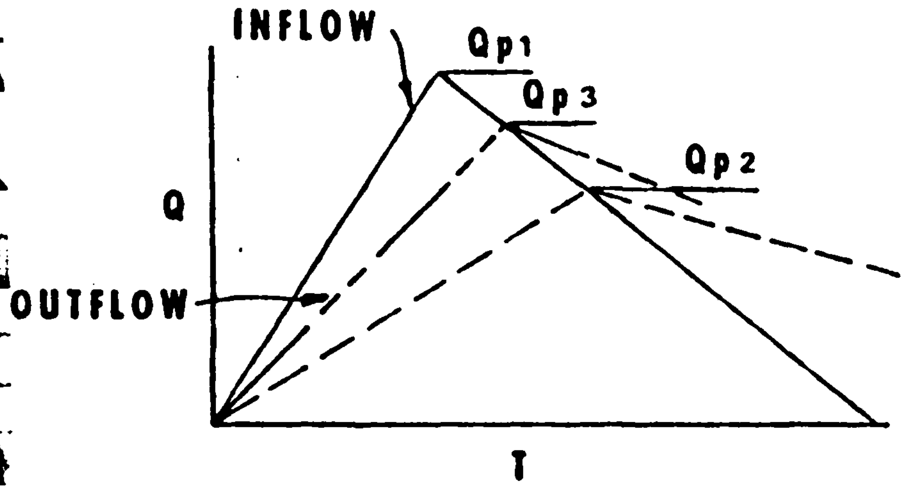
UNCLASSIFIED F/G 13/13 NL

END
DATE
FILMED
8-84
DTIC



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

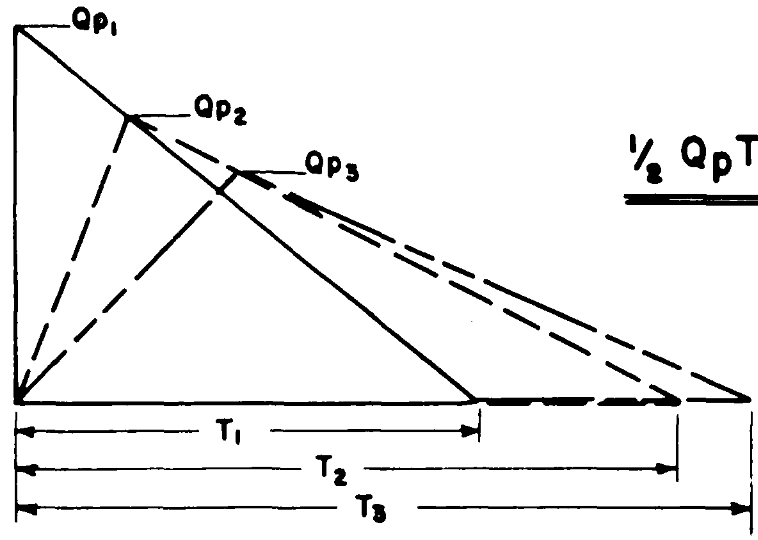
c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".

"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\tfrac{1}{2}\, Q_p T = 12\, S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27}\, W_b \sqrt{g}\, Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{ave}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

Cahn Engineers Inc. **Consulting Engineers**

Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND Sheet 1 of 11
Computed By HLE Checked By CKL Date 1/22/79
Field Book Ref. Other Refs. CE#27-575-KA Revisions

HYDROLOGIC / HYDRAULIC INSPECTION

UNION POND DAM, MANCHESTER, CT.

I) PERFORMANCE AT TEST FLOOD CONDITIONS:

1) MAXIMUM PROBABLE FLOOD:

a) WATERSHED CLASSIFIED AS "ROLLING"

b) WATERSHED AREA: DA = 53.9 SQ MI (USGS, HARTFORD OFFICE)

c) FROM NED-ACE "PRELIMINARY GUIDANCE FOR ESTIMATING MAX. PROBABLE DISCHARGES" - GUIDE CURVE FOR PMF - PEAK FLOW RATES:

$PMF \approx 1150$ CFS/SQ MI

d) PEAK INFLOW: $PMF = 1150 \times 53.9 \approx 62000$ CFS

2) SPILLWAY DESIGN FLOOD (SDF):

a) CLASSIFICATION OF DAM ACCORDING TO NED-ACE RECOMMENDED GUIDELINES.

i) SIZE*: STORAGE (MAX) ≈ 720 AC-FT $(50 \leq S \leq 1000$ AC FT$)$,
HEIGHT $\approx 33'$ $(25 \leq H \leq 40$ FT$)$

*STORAGE: FROM U.S. INVENTORY OF DAMS DATED 11/26/73; STORAGE AT FLOWLINE: 515 AC FT, AT MAX. POOL 565 AC FT; HOWEVER C.E. ROUGH CHECK BASED ON LAKE AREA FROM CONN. DEP, WATER & RELATED RESOURCES INVENTORY SHEET OF A=51.5 AC. AND SPILLWAY CREST TO TOP OF DAM DEPTH OF 4' (A.C. RISE ENGRS, WORCESTER, MASS.): $S_{MAX} \approx 515 + 4 \times 51.5 \approx 720$
HEIGHT: ESTIMATED FROM A.C. RISE ENGRS. DRAWING No 1401-C "GATE HOUSE AND DAM" DATED JULY 27, 1901.

Cahn Engineers Inc. **Consulting Engineers**

Project NON-FEDERAL DAMS INSPECTION Sheet 2 of 11
Computed By [initials] Checked By [initials] Date 1/22/79
Field Book Ref Other Refs. CE#27-575-KA Revisions

UNION POND DAM

2,a - (Cont'd) CLASSIFICATION

ii) HAZARD POTENTIAL: THE DAM IS LOCATED U/S OF URBANIZED PORTION OF MANCHESTER, CT. - SOME LOW GROUND HOUSES / INDUSTRIAL BUILDINGS ARE LOCATED NEAR ADAMS ST. (±) 1.5 MILES D/S. DEPENDING ON THE MAGNITUDE OF A POSSIBLE FLOOD WAVE, OTHER STRUCTURES, CLOSER TO THE DAM MAY ALSO BE AFFECTED.

iii) CLASSIFICATION:

SIZE: SMALL

HAZARD: HIGH

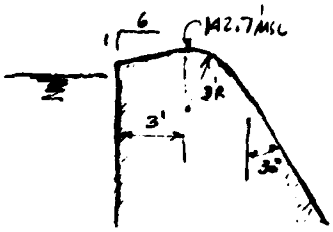
b) SDF = PMF = 62000 CFS   1/2 PMF = 31000 CFS

3) SURCHARGE AT PEAK INFLOWS:

a) PEAK INFLOW: $Q_{P_1} = 62000$ CFS $Q'_{P_1} = \frac{1}{2}$ PMF $= 31000$ CFS

b) SPILLWAY (OUTFLOW) RATING CURVE

i) SPILLWAY.



THE SPILLWAY IS CLASSIFIED AS A BROAD CRESTED COMPOUND WEIR OF TRAPEZOIDAL CROSS SECTION WITH INCLINED FACES. THE U/S FACE ON (±) 6H TO 1V SLOPE AND THE D/S FACE AT 30° WITH THE VERTICAL ((±)1H TO 1.75V) (A.C. RISE, ENGINEER, WORCESTER, MASS. DWG Nº 1401-C "GATE HOUSE AND DAM FOR CHENEY BROS." DATED JUL. 27, 1901). IN PLAN, THE SPILLWAY IS "L" SHAPED; ONE LEG ARCHED (±) 104' (A.C. RISE DWG. ADJUSTED

Cahn Engineers Inc. **Consulting Engineers**

Project NON-FEDERAL DAMS INSPECTIONS Sheet 3 of 11
Computed By HU Checked By CKJ Date 1/23/79
Field Book Ref. Other Refs. CE #27-515-KA Revisions

UNION POND DAM

3,b-Cont'd) OUTFLOW RATING CURVE

ROUGHLY TO ACTUAL LENGTH BY C.E. FIELD OBSERVATIONS) AND THE OTHER, STRAIGHT, (±)194' LONG. THEREFORE, THE TOTAL SPILLWAY CREST IS (±)300' LONG. THE HEIGHT BETWEEN THE SPILLWAY CREST (ELEV. 142.7' MSL)* AND THE TOP OF THE DAM (ELEV. 146.7' MSL) IS H=4'. THE U/S VERTICAL DEPTH OF THE SPILLWAY FACE IS P=8'

∴ SPILLWAY DISCHARGE COEFFICIENT, ASSUME: C=3.5

USING THE CREST ELEVATION AS DATUM (ELEV. 142.7' MSL), THE SPILLWAY DISCHARGE IS APPROXIMATED BY:

$$Q_S = 1050\,H^{3/2}$$

ii) EXTENSION OF RATING CURVE FOR SURCHARGE HEADS ABOVE TOP OF DAM.

BESIDES THE CONCRETE ABUTMENTS AT BOTH SIDES OF THE SPILLWAY, TO THE RIGHT SIDE BANK AND TO THE GATE HOUSE AT THE LEFT, THE DAM EXTENDS AS AN EARTH DIKE, TO THE LEFT BETWEEN THE GATE HOUSE AND THE LEFT SIDE BANK. THE DIKE WRAPS BEHIND THE GATE HOUSE TO TIE WITH THE SPILLWAY LEFT ABUTMENT
THE TOP ELEV. OF THE DIKE WILL BE ASSUMED THE SAME ELEV. 146.7' MSL OF THE SPILLWAY ABUTMENTS IN THESE COMPUTATIONS. (DIKE LOW POINTS ASSUMED REPAIRED TO CORRECT ELEV.)
THE SPILLWAY RIGHT ABUTMENT IS (±)6' WIDE × 37' LONG; THE LEFT ABUTMENT IS (±) 6' WIDE × 20' LONG. THE EARTH DIKE IS ±10' WIDE

*NOTE: A.C. RISE MAP ELEVS. TRANSFERED TO MSL DATUM BY C.E. SURVEY FROM CITY OF MANCHESTER, DEPT. OF TRANSPORTATION, BM AT TRANSFORMER PAD TO THE LEFT OF THE RESERVOIR (SEE C.E. DWGS)

Cahn Engineers Inc.   Consulting Engineers

Project NON-FEDERAL DAMS - INSPECTION   Sheet 4 of 11
Computed By HLL   Checked By CRG   Date 1/23/79
Field Book Ref   Other Refs CE#27-545-KA   Revisions

UNION POND DAM

3, b- Cont'd) OUTFLOW RATING CURVE

AT THE TOP AND 1.5^H TO 1^V SIDE SLOPES AND IS (±) 180' LONG.

THE TERRAIN BEYOND THE RIGHT SIDE ABUTMENT RISES (±) IN A 3^H TO 1^V SLOPE TO (±) 14' ABOVE THE DAM. THE TERRAIN AT THE LEFT OF THE DIKE RISES GRADUALLY (±) 24' IN A DISTANCE OF (±) 900'.

ASSUME C = 3.0 FOR THE ABUTMENTS AND DIKE AND
C = 2.5 FOR THE OVERFLOW AT THE SIDES OF THE DAM.

ASSUME ALSO, EQUIVALENT LENGTHS FOR THE TERRAIN AT THE SIDES OF THE DAM AS FOLLOWS:

$$L'_R = \frac{2}{3}\left(\frac{3}{1}\right)(H-4) = 2(H-4)$$

$$L'_L = \frac{2}{3}\left(\frac{900}{24}\right)(H-4) = 24(H-4)$$

THE TOTAL OUTFLOW RATING CURVE CAN BE APPROXIMATED BY:

$$Q = 1050H^{3/2} + 710(H-4)^{3/2} + 65(H-4)^{5/2}$$

THE OUTFLOW RATING CURVE IS PLOTTED ON NEXT PAGE.

c) SPILLWAY CAPACITY TO TOP OF DAM:

$H = 4'$ ∴ $Q \doteq 8400^{CFS}$ (13.5% of Q_P; 27% of Q'_P)

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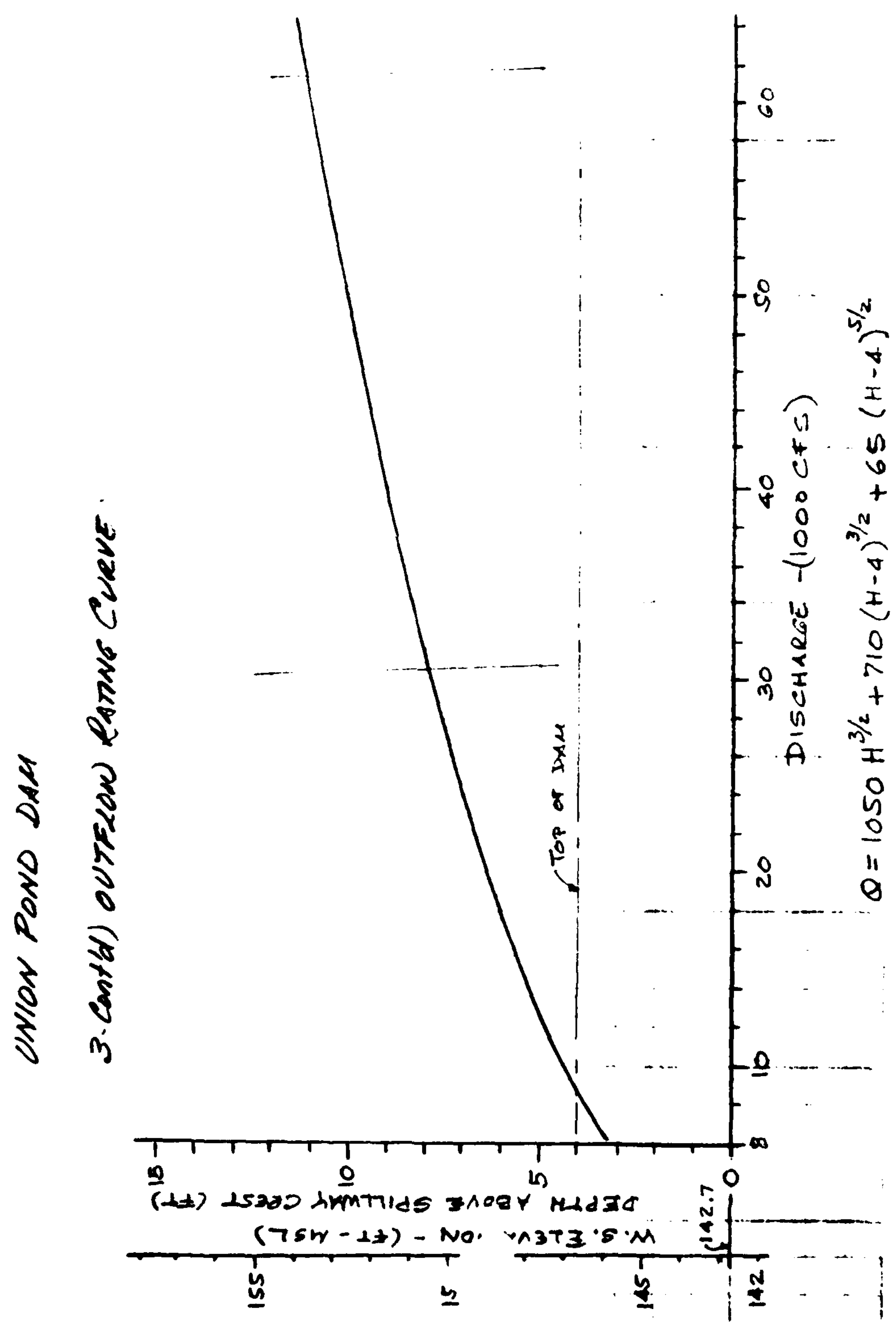

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UNION POND DAM

3-Cont'd) SURCHARGE AT PEAK INFLOWS

d) SURCHARGE HEIGHT TO PASS Q_{P_1}:

i) @ Q_{P_1} = PMF = 62000^{CFS} $H_1 \approx 11.2'$

ii) @ $Q_{P_1}' = \frac{1}{2}$PMF = 31000^{CFS} $H_1' \approx 7.9'$

4) EFFECT OF SURCHARGE STORAGE ON MAX. PROBABLE DISCHARGES (OUTFLOW)

a) RESERVOIR (POND) AREA @ FLOW LINE: $^*A_0 = 51.5^{AC.}$

*FROM CONN. D.E.P. WATER & RELATED RESOURCES - INVENTORY SHEET. C.E. CHECK MEASURE (USGS. 1:24000) $A \approx 51.4^{AC}$; CONTOUR 4' ABOVE W.L. $A \approx 71^{AC}$

∴ ASSUME AVE. LAKE AREA WITHIN EXPECTED SURCHARGE, $A = 60^{AC}$.

b) ASSUME NORMAL POOL LEVEL AT SPILLWAY CREST (ELEV. 142.7 MSL)

c) WATERSHED AREA: D.A = 53.7$^{SQ.MI}$ (SEE P.1)

d) DISCHARGE Q_{P_2} AT VARIOUS SURCHARGE ELEVATIONS:

$H = 12'$ $V = 60 \times 12 = 720^{AC\text{-}FT}$ ∴ $S = \frac{720}{53.7 \times 53.3} = 0.25''$

$H = 5'$ $V = 300^{AC\text{-}FT}$ ∴ $S = 0.10''$

∴ FROM APPROXIMATE STORAGE ROUTING NED-ACE GUIDELINES (19" MAX. PROBABLE R.O. IN NEW ENGLAND):

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UNION POND DAM

A,d-Cont'd) DISCHARGE (Q_{P_2}) AT VARIOUS SURCHARGE ELEVS.

$Q_{P_2} = Q_{P_1}\left(1 - \frac{S}{19}\right)$ AND FOR 1/2 PMF: $Q'_{P_2} = Q'_{P_1}\left(1 - \frac{S}{9.5}\right)$

∴ FOR:

$H = 12'$ $Q_{P_2} \cong 61200^{CFS}$ $Q'_{P_2} \cong 30200^{CFS}$

$H = 5'$ $Q_{P_2} \cong 61700^{CFS}$ $Q'_{P_2} \cong 30700^{CFS}$

e) PEAK OUTFLOW (Q_{P_3})

USING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING ALTERNATE" METHOD (SEE p.5)

$Q_{P_3} \cong 61200^{CFS}$ $H_3 \cong 11.1'$ FOR Q_{P_1} = PMF

$Q'_{P_3} \cong 30500^{CFS}$ $H'_3 \cong 7.9'$ FOR $Q'_{P_1} = \frac{1}{2}$ PMF

f) SPILLWAY CAPACITY RATIO TO OUTFLOW:

SPILLWAY CAPACITY TO TOP OF DAM: $Q_S \cong 8400^{CFS}$

∴ SPILLWAY CAPACITY IS (±) 14% THE OUTFLOW @ PMF AND (±) 28% THE OUTFLOW @ 1/2 PMF.

5) SUMMARY:

a) PEAK INFLOW Q_{P_1} = PMF = 62000^{CFS} Q'_{P_1} = 1/2 PMF = 31000^{CFS}

b) PEAK OUTFLOW $Q_{P_3} = 61200^{CFS}$ $Q'_{P_3} = 30500^{CFS}$

c) SPILLWAY MAX. CAPACITY: Q_S: 8400^{CFS} OR 14% OF Q_{P_3} AND 28% OF Q'_{P_3}

THEREFORE, AT SDF = 1/2 PMF, THE DAM IS OVERTOPPED (±) 3.9' (WS.EL. 150.6) OR, TO AN APP. SURCHARGE ABOVE THE SPILLWAY CREST OF (±) 7.9'.

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UNION POND DAM

II) DOWNSTREAM FAILURE HAZARD

1) PEAK FLOOD AND STAGE IMMEDIATELY D/S FROM DAM.

a) BREACH WIDTH:

i) MID-HEIGHT (±) ELEV. 130' MSL $(146.7 - \frac{33}{2} = 130.2$ SAY, 130' MSL)

ii) APPROX. MID-HEIGHT LENGTH $\ell \simeq 310'$ (FROM A.C. RICE DWG & C.E. AUTOLEVEL SEE ¶ 3.b.i pp. 2-3)

iii) BREACH WIDTH (SEE NED-ACE D/S DAM FAILURE GUIDELINES)

$W = 0.4 \times 310 = 124'$ ∴ ASSUME $W_b = 120'$

b) PEAK FAILURE OUTFLOW (Q_p)

ASSUME SURCHARGE TO TOP OF DAM; THEREFORE,

i) HEIGHT AT TIME OF FAILURE: $Y_0 = 33'$

ii) SPILLWAY DISCHARGE: $Q_s = 8400 - 3400 = 5000$ CFS.

NOTE: THE DAM IS MOSTLY SPILLWAY AND THEREFORE, THE SPILLWAY FLOW HAS BEEN REDUCED BY THE FLOW OVER THE PORTION OF "BREACHED" SPILLWAY ($L \simeq 120'$).

iii) BREACH OUTFLOW (Q_b):

$$Q_b = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2} \simeq 38200 \text{ CFS}$$

iv) PEAK FAILURE OUTFLOW (Q_p): $Q_p = Q_s + Q_b = 41600$ CFS

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UNION POND DAM

1-(Cont'd) PEAK FLOOD AND STAGE IMMEDIATELY D/S FROM DAM.

c) FLOOD WAVE HEIGHT IMMEDIATELY D/S OF DAM:

$$Y \doteq 0.44 Y_0 \doteq \underline{15'}$$

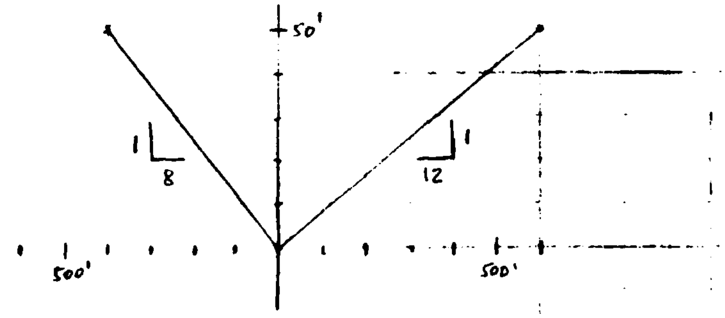
2) ESTIMATE OF D/S DAM FAILURE CONDITIONS AT IMPACT AREA.

(SEE NED-ACE GUIDELINES FOR ESTIMATING D/S DAM FAILURE HYDROGRAPHS).
ASSUME RESERVOIR FULL TO TOP OF DAM AT TIME OF FAILURE.

a) RESERVOIR STORAGE AT TIME OF FAILURE: $S \doteq 720$ AC-FT (SEE P. 1)
$S/2 \doteq 360$ AC-FT.

b) TYPICAL D/S CROSS SECTION & RATING CURVES.

(FROM USGS MANCHESTER, CT. QUADRANGLE SHEET, PHOTOREVISED 1965, SCALE 1:24000)



ASSUME: i) $n = 0.050$
ii) SLOPE: $S_0 = 0.42\%$ (DROPS 30' IN (±) 7200')
$\therefore S^{1/2} = 0.065$

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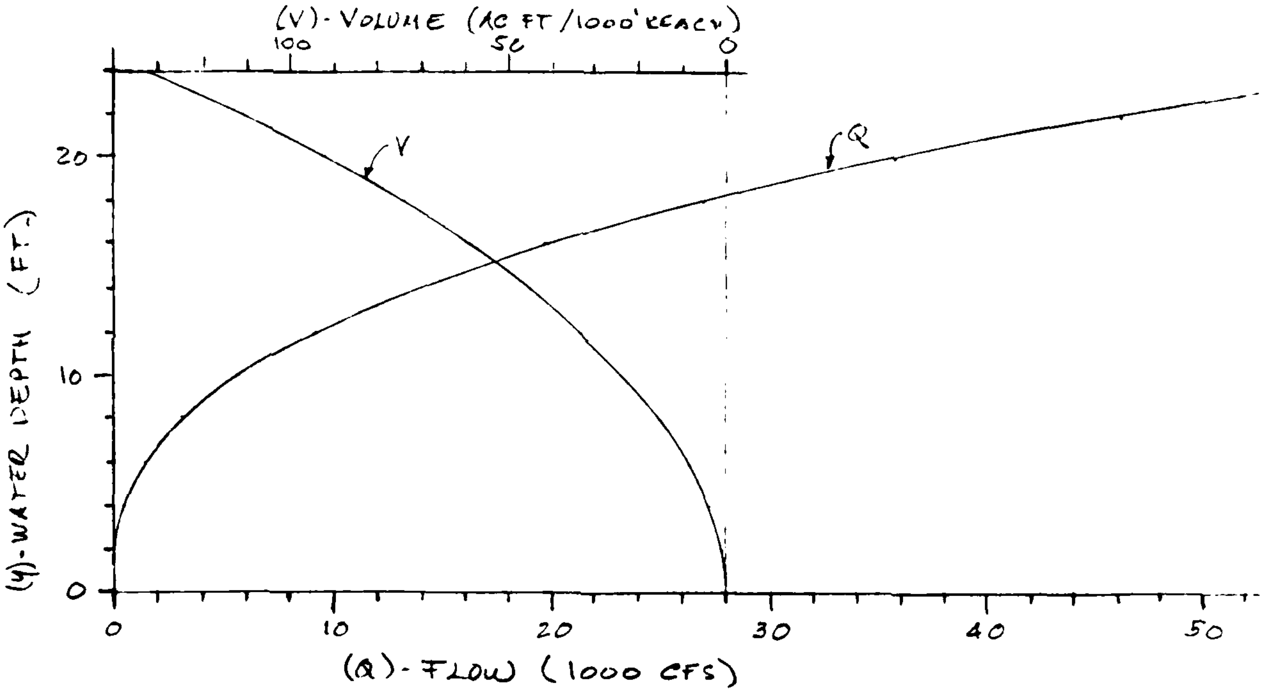
UNION POND DAM

2-Cont'd) D/S CROSS SECTION - RATING CURVE:

C) RATING CURVES



d) REACH OUTFLOW (Q_{P_3})

i) ASSUME REACH LENGTH $L = 8000'$ (± ADAMS ST. - MANCHESTER, ...)

ii) @ $Q_{P_1} = 41600^{CFS}$ ∴ $y_1 \approx 21.2'$ (FROM RATING CURVE)

∴ $V_1 \approx 824^{AC\,FT} > \frac{S}{2}$ ($\frac{S}{2} \approx 360^{AC\,FT}$)

∴ iii) TRY NEW REACH LENGTH · $L_1 = 3500'$ (REACH No. 1)

iv) @ $Q_{P_1} = 41600^{CFS}$; $y_1 \approx 21.2'$ ∴ $(V_1)_1 \approx 360^{AC\,FT} = \frac{S}{2}$ ok

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UNION POND DAM

2, d - (Cont'd) REACH OUTFLOW ($S/_3$)

v) $(Q_{P_2})_1 = Q_{P_1}\left(1 - \frac{V_1}{S}\right) \approx 20800^{CFS}$ ∴ $(Y_2)_1 \approx 16.3'$ $(V_2)_1 \approx 214^{AC\cdot FT}$

vi) AVE. VOLUME IN REACH No. 1 : $(V_{AVE})_1 = 287^{AC\cdot FT}$

vii) REACH No 1 OUTFLOW: $(Q_{P_3})_1 \approx 25000^{CFS}$ $(Y_3)_1 \approx 17.5'$

viii) ASSUME LENGTH OF REACH No. 2 $L_2 = 4500'$

ix) $(Q_{P_1})_2 \approx 25000^{CFS}$ $(Y_1)_2 \approx 17.5'$ $(V_1)_2 \approx 316^{ACFT} < \frac{S}{2}$ ok

x) $(Q_{P_2})_2 \approx 14000^{CFS}$ $(Y_2)_2 \approx 14.1'$ $(V_2)_2 \approx 205^{AC\ FT}$

xi) AVE. VOLUME IN REACH No. 2 : $(V_{AVE})_2 = 260^{AC\cdot FT}$

xii) REACH No. 2 OUTFLOW: $(Q_{P_3})_2 \approx \underline{16000}^{CFS}$ $(Y_3)_2 \approx 14.8'$; SAY, $\underline{15'}$

3) SUMMARY

a) PEAK FAILURE OUTFLOW: $Q_{P_1} \approx 41600^{CFS}$
b) REACH OUTFLOW (IMPACT AREA) $Q_{P_3} \approx 16000^{CFS}$
c) AVE. WATER DEPTH (STAGE) $Y_3 \approx 15'$ (AT IMPACT AREA)

APPENDIX

SECTION E: INVENTORY OF DAMS IN UNITED STATES

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR. DIST.	STATE	COUNTY	CONGR. DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	13	NED	CT	003	01				UNION POND DAM	4148.0	7231.7	09FEB79

POPULAR NAME	NAME OF IMPOUNDMENT
	UNION POND

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	05	HOCKANUM RIVER	MANCHESTER	1	49200

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUC. HEIGHT (FT.)	HYDRAU. HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
RECTERGBPG	1972	R	35	33	720	515	NED	N	N	N	N	09FEB79

REMARKS
22 OLD DAM RAISED TO PRESENT HEIGHT IN 1901 REHABILITATED IN 1972

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NO.	NAVIGATION LOCKS LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
1	590	U	298	8400												

OWNER	ENGINEERING BY	CONSTRUCTION BY
TOWN OF MANCHESTER	A C RICE	

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NONE	NONE	NONE	NONE

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
CAHN ENGINEERS INC	27NOV78	PUBLIC LAW 92-367 8AUG1972

REMARKS